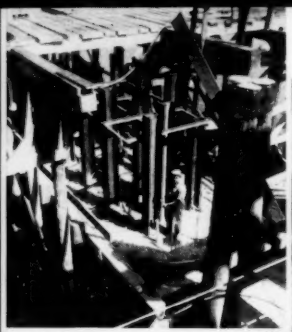
ACI News Letter

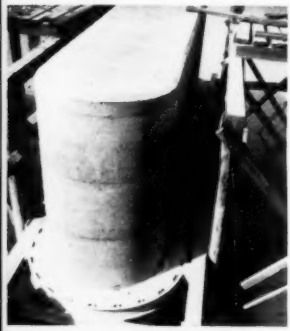
VOL. 23 No. 7 JOURNAL of the AMERICAN CONCRETE INSTITUTE March 1952



Underwater section of PREPAKT H-Pile pier under construction. The H-piles are in position and the form is filled with coarse aggregate ready for Intrusion grouting.

General view of piers for the Stave River Bridge, Vancouver, B. C., showing temporary bridging between new piers, which permitted direct dumping of coarse aggregate into pier forms.

Above-water section of pier showing form filled with coarse aggregate ready for grouting. These forms were filled directly from dump trucks.



Finished pier structure showing steel sheeting originally used as a cofferdam and concrete form, and retained as part of the finished pier.

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MEANS STRENGTH • SPEED • ECONOMY

Prepakt methods and materials produce superior results in the construction of bridge piers and other concrete structures—superior results as measured by pier strength, material economy, and speed in construction.

Another case history proving this point is found in this highway bridge in British Columbia where the piers and abutments were installed with Intrusion-Prepakt methods. Test cores, drilled upon completion of the piers, showed a concrete compressive strength of 4,000 psi. at 28 days—an assurance of 90-day strength of at least 6,000 psi. *Only with Prepakt could such concrete strength be obtained with so little cement—4 bags per cubic yard*—and with shrinkage so low as to be negligible.

Add to this the inherent resistance of Prepakt Concrete to deterioration from freezing and thawing action and you have the logical answers to why Prepakt is best for concrete structures requiring the best.

Full information on Prepakt as used in this and other structures may be had by calling or writing the Main Office, Cleveland, O.

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ACI

News Letter Contents

March 1952

* On the Cover—Industrial building in Canada constructed of precast vacuum concrete sandwich-type walls and precast roof. For further details see paper, p. 581, this issue.

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48th Annual Convention Draws Large Registration

Cincinnati cordially welcomed 654 members and friends of the American Concrete Institute attending its 48th annual convention February 26, 27 and 28 at the Netherland Plaza Hotel. Registration far exceeded expectations due largely, it is believed, to efforts of the local committee in publicizing the event. A crowded technical program attracted large audiences at each session.

Highlight of the convention was the luncheon on Thursday noon in honor of Harvey Whipple who retired as secretary-treasurer after 32 years. Ten past-presidents were there to pay their respects. Stanton Walker presided and made the introductions. Herbert J. Gilkey paid tribute to Mr. Whipple's long service and announced that a gift was to be purchased with contributions from friends and delivered to the Whipple home. About 200 attended the luncheon, many of them ACI old-timers who, as Professor Gilkey said, "remembered Harvey Whipple of the Windsor tie."

New officers elected were: A. T. Goldbeck, president; Charles H. Scholer, vice-president; and A. Allan Bates, J. W. Kelly, F. N. Menefee and Bailey Tremper, directors.

Retiring Board members received Certificates of Appreciation: Robert F. Blanks, Harmer E. Davis, Harry Delzell, N. M. Newmark, R. W. Spencer and Harvey Whipple.

Institute awards were presented at the luncheon on Wednesday noon. The Henry C. Turner Medal, last presented in 1947, was awarded to Raymond E. Davis "in recognition of numerous outstanding and greatly diversified contributions to the technology of concrete as a material"; the Alfred E. Lindau Award was presented to Douglas E. Parsons "for his many years of quiet perseverance in directing and coordinating development of standards of practice for reinforced concrete design"; the Leonard C. Wason Medal for the "most meritorious paper" was given to Walter H. Price; the Wason Medal for Noteworthy Research, to W. J. McCoy and A. G. Caldwell; Otto Safir, Vancouver, B. C., was unable to be present to receive the ACI Construction Practice Award.

Technical subjects varied widely; Prestressed Concrete, Curing, Durability, Ultimate Load Design, Materials and Properties; Design, Construction, Research.

The convention closed on Thursday afternoon with a panel session over which Professor Gilkey presided with his usual *savoir faire.*

The complete convention story will be published in the April News Letter.

First Annual Minnesota Concrete Conference

By Fred R. McComb*

More than 300 contractors, architects, engineers, ready mixed concrete and aggregate producers, cement finishers and other construction personnel from Minnesota and surrounding states attended the first annual concrete conference at the Center for Continuation Study, University of Minnesota, Minneapolis, December 3 and 4, 1951. Quality concrete was the theme of the two-day program conducted by the University of Minnesota and the Portland Cement Assn. Organizations which cooperated to make the conference an outstanding success included: The Associated General Contractors of Minnesota, The Northwest Section of ASCE, The Minnesota Society of Architects, the North Central Commercial Aggregate and Ready Mixed Concrete Producers Assn., the Minnesota Society of Professional Engineers and local members of the American Concrete Institute.

Stanton Walker, Director of Engineering, National Sand and Gravel Assn. and National Ready Mixed Concrete Assn., Washington, D. C., and Past President of ACI, was the featured speaker of the conference with his paper, "Application of Quality Concrete Principles to Ready Mixed Concrete."

The thorough discussion sessions covered the types and uses of portland cement; characteristics and standards for concrete aggregates; fundamentals quality concrete; air entrainment; testing and local problems; placing, finishing, curing, protection and forms for concrete; and ready-mixed concrete—plant and transmit mix.

Other speakers who took part in presenting information and instruction under the above topic headings were Fred R. McComb, district engineer, and R. W. Randall, district structural engineer, both of Portland Cement Assn., Minneapolis; John Swanberg, engineer of materials and research, Minnesota State Highway Dept.; Prof. J. A. Wise, Civil Engineering Dept., University of Minnesota; C. D. Bullock, regional structural engineer, Portland Cement Assn., Kansas City, Mo.; Charles W. Britzius, president, Twin City Testing and Engineering Laboratory, St. Paul; John H. Banker, construction superintendent, Portland Cement Assn., Chicago; John Ellison, construction superintendent, Fegles Construction Co., Ltd., Minneapolis; Earl E. Eckert, superintendent of construction and engineering, George A. Hormel Company, Austin, Minn.; Richard W. Teague, construction superintendent, Minneapolis Water Dept.; Phil Woog, Jr., Cement Finishers Local No. 20, St. Paul; George Shepard, chief engineer, City of St. Paul; N. T. Rykken, Whitney's Ready Mixed Concrete Co., Duluth; and R. T. Lindh, Twin City Ready Mix Concrete Co., Minneapolis.

Presiding over the various discussion periods were Walter H. Wheeler, consulting engineer, Minneapolis; Prof. T. W. Thomas, Civil Engineering Dept., University of Minnesota; Roy Shelgren, St. Paul contractor; and D. S. Haarstick of Haarstick, Lundgren and Associates, St. Paul architects.

*District Engineer, Portland Cement Assn. Minneapolis, Minn.

Who's Who This Month

J. Calleja

J. Calleja, head of physical chemistry department, Instituto Tecnico de la Construccion y del Cemento, Madrid, Spain, is the author of "New Techniques in the Study of Setting and Hardening of Hydraulic Materials," p. 525.

J. J. Shideler

J.J. Shideler, material engineer, U.S. Bureau of Reclamation, Denver, Colo., writes about "Calcium Chloride in Concrete," p. 537.

Mr. Shideler joined the Bureau of Reclamation in 1939 and has been associated with investigations of thermal and physical properties of concrete for Bureau structures, as well as with electrical gaging, testing and control equipment used in the laboratory.

He received a BS in electrical engineering from the University of Denver in 1937. He is a registered engineer in Colorado, a member of AIEE, the Colorado Society of Engineers and has been an ACI member since 1947.

K. Hajnal-Konyi

K. Hajnal-Konyi, consulting engineer, London, England, came to the Cincinnati convention personally to present his paper "Developments in Reinforcing Steel and Concepts of Safety," p. 561.

Dr. Hajnal-Konyi was employed in the bridge department of the Gutehoffnungshutte, Sterkrade Rheinland, 1921-1923, and joined Prof. A. Kleinlogel, Darmstadt, in 1924, with whom he was chief engineer for 12 years.

Since 1936, he has been a consulting engineer in London, designing reinforced structures for many factories, schools and apartments. His main interests are the development of shell concrete structures, prestressed concrete and the application of high-tensile steel to reinforced concrete. In the latter field he has done extensive research.

Born in Budapest, Hungary, Dr. Hajnal-Konyi was graduated from the Federal Polytechnic Institute, Zurich, Switzerland, in 1921 and obtained his degree as Doctor of Engineering from Technical University, Darmstadt, Germany. He has been a member of ACI since 1936.

K. P. Billner

"Applications of Vacuum Concrete," p. 581, are described by K. P. Billner, president, Vacuum Concrete, Inc., Philadelphia, Pa.

Awarded the Franklin Institute John P. Brown medal in 1947 for his work with vacuum processing of concrete, his name is also identified with such inventive contributions to concrete engineering as Aerocrete, the use of aluminum powder in concrete to improve bond to steel, vacuum concrete, interlocked helical reinforcement (sometimes referred to as "bed spring" reinforcing), electric prestressing and mechanical prestressing.

A graduate of the Chalmers Institute of Technology, Goteborg, Sweden, Mr. Billner has been active in engineering and building construction in Sweden and the United States. He is the founder of Vacuum Concrete, Inc., and is a director of the Aerocrete Corp. of America and organized similar concerns in Canada and England. He is a member of ACI Committee 324, Precast Reinforced Concrete Structures, and has been affiliated with ACI since 1936.

Thompson-Starrett Co., Inc., to construct dam in Turkey

The Turkish Government has awarded an $11,000,000 contract to Thompson-Starrett Co., Inc., to head a group of Turkish-German associates to construct a huge dam on the Sakarya River, the first major part of its scheduled $35,000,000 Sariyar hydroelectric project designed by American engineers to help accelerate the industrialization of northwestern Turkey. ECA funds are being utilized to build the new hydroelectric project.

Thompson-Starrett is best known for building many skyscrapers, most notable of which is the Woolworth Building and Waldorf Astoria Hotel. However, with a group of associates it also built the $78,000,000 Grand Coulee Dam.

Sariyar Dam will be the largest dam in Turkey and will require about 700,000 cu yd of concrete.

Positions and Projects — ACI Members

Lewis elected to executive committee

Carlton J. Lewis, secretary and general manager of the Buffalo Gravel Corp., Buffalo, N. Y., has been elected to the executive committee of the General Contracting Employers' Association of Buffalo.

Mercer extends activities

The April 1951 *News Letter* outlined some of the extensive travels and varied activities of L. Boyd Mercer in the field of concrete technology during the past 20 years. Since then he has again been on the move—New Zealand, Canada, United States, England, and France, visiting laboratories and observing concrete developments from the soniscope to prestressing.

The principal purpose of Mr. Mercer's visit to London, where he spent two months, was to extend activities of Ready Mixed Concrete Ltd. from Australia to England. This is an interesting reversal of the usual procedure by which British industries generally develop and extend their activities from England to the Dominions.

Returning to Australia, Mr. Mercer found himself submerged with letters from all corners of the world, from India to Peru, following the appearance of reviews (ACI JOURNAL, Dec. 1951, p. 360) of his publication, "The Law of Grading for Concrete Aggregates." He says that the moral to be learned from his concrete activities of the past year is that if you do concrete research in your spare time there is no spare time.

Turner Construction Co. elections

Dr. Howard S. Turner, director of the Research & Development Division of Pittsburgh Consolidation Coal Co., Pittsburgh, Pa., was elected a member of the board of directors of the Turner Construction Co. recently. Dr. Turner is a son of the late J. Archer Turner, president of the firm from 1941 to 1946.

Donald C. Andrews, vice president in charge of New York operations, was also elected a director, while Edward A. Courter, who has charge of the Chicago office of the Turner Construction Co., was made a vice president.

Miller appointed sales engineer

Thomas C. Miller, formerly supervisor, Masonry Materials Section, Armour Research Foundation, Illinois Institute of Technology, Chicago, has been appointed executive sales engineer in the industrial division of National Gypsum Co., Buffalo, N. Y. Mr. Miller, who has had 20 years of experience in the lime and portland cement industries, will concentrate on the company's lime activities. He has had extensive experience in the design, construction and operation of shaft lime kilns and in the production of chemical lime for specific purposes as well as in the manufacture of high calcium and dolomitic finishing and masonry mortar limes. He also has had experience in the development of lightweight concrete aggregates from clay and shale.

Condensation control in buildings

A conference on the relation of paints, papers and insulating materials to condensation control in buildings was held Feb. 26 and 27 by the Building Research Advisory Board at the National Academy of Sciences, Washington, D. C. The practical and technological aspects of condensation; mechanics of moisture movement; paint, paper and foil films; and insulation and construction problems were discussed.

Julian Hinds

Julian Hinds retired from his long-time job as general manager and chief engineer of the Metropolitan Water District of Southern California. Presently he is general manager and chief engineer of the United Water Conservation District.

Dreyer appointed vice-president and chief engineer

Walter Dreyer has been appointed vice-president and chief engineer of the Pacific Gas and Electric Co., San Francisco, Calif. Mr. Dreyer received his BS degree in civil engineering at the University of California in 1916 and has been associated with the Pacific Gas and Electric Co. since that time. He has been affiliated with ACI since 1922.

Dewey and Almy names director

Robert T. Haslam has been elected a member of the board of directors of the Dewey and Almy Chemical Co., Cambridge, Mass., Bradley Dewey, president, has announced.

Mr. Haslam, a widely known chemical and marketing executive, is a former vice-president and director of Standard Oil Co. (N. J.) where his responsibilities included direction of research and sales.

Prior to joining Standard Oil in 1927, Mr. Haslam was professor of chemical engineering at Massachusetts Institute of Technology and is now a member of the corporation of MIT.

Becker plays important part in St. Louis building

As past chief engineer of bridges and building in St. Louis, a few of William C. E. Becker's building achievements include the Homer G. Phillips Hospital, City Hospital, Koch Hospital, Jewel Box, City Sanitorium, Aloe Plaza and Armory Building—more than $12½ million worth of construction.

In 1931 he was appointed chief engineer, Building Dept. of St. Louis, and three years later he was appointed city building commissioner.

At present Mr. Becker is in consulting engineering practice. A few of his recent jobs include Anheuser-Busch, Washington and Missouri Universities, Jefferson Barracks Hospital, and Scullin Steel.

Short course

A short course on ready mixed concrete design and control was held at Stillwater, Okla., Jan. 28-30, under the direction of the School of Civil Engineering and the Division of Engineering and Industrial Extension of Oklahoma A. and M. College. A committee representing the ready mixed concrete industry was active in planning the course. Topics discussed were tilt-up construction, concrete construction from the ready mixed concrete operators viewpoint, safety and accident prevention, theory of proportioning mixes and use of admixtures. Methods of making trial mixes were demonstrated.

Furlong transferred

W. C. Furlong, service engineer, Hawkeye-Marquette Portland Cement Co., Des Moines, Iowa, has been transferred to the St. Louis office of Marquette Cement Mfg. Co. as service engineer in Missouri and southern Illinois.

Reiter elected executive vice-president

George H. Reiter has been elected executive vice-president of Universal Atlas Cement Co., New York, N. Y. He was formerly vice-president and general sales manager.

Spencer appointed associate professor

Ernest L. Spencer, formerly assistant professor of civil engineering, Northeastern University, Boston, Mass., has been appointed associate professor. Professor Spencer is in direct charge of the concrete laboratory which is used for instructional and research purposes at the University.

National Building Code changes

Amendments to the National Building Code of the National Board of Fire Underwriters may pave the way for more open-air parking garages. The NBFU Code now allows such garages to be built more economically than the fire resistive construction formerly required under the 1949 Code.

The new construction types have been recognized because parking garages have had a favorable fire record and the open sides of open air parking garages make it easy for firemen to get at the fire. The open sides also allow gases and smoke to escape from the building.

Another change in the Code concerns standards on reinforced concrete. The new type A 305 steel bar for reinforcing concrete is now recognized which will reduce construction costs and also conserve steel. This change adopts the 1951 revision of the ACI Building Code.

Copies of the National Building Code and of the leaflet containing the new amendments may be obtained on application at the National Board of Fire Underwriters which has offices at 85 John St., New York 38; 222 West Adams St., Chicago 6; and Merchants Exchange Bldg., San Francisco 4.

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Perlite Institute names secretary

J. John Brouk, president and chairman of the Board of Directors of the Perlite Institute, has announced that the Board has employed Richard L. Davis as secretary-treasurer, effective February 15. Mr. Davis has been secretary of the Industrial Mineral Wool Institute for the past 11 years.

Turner Construction Co. has record year

The Turner Construction Co., New York, completed $100,221,210 worth of construction in 1951, the largest volume of work in its history, H. C. Turner, Jr., president, announced in the 50th annual report to stockholders. Contract earnings also exceeded those in any previous year but net earnings after taxes were only slightly larger than in 1950 due to heavier taxes. The company will observe the 50th anniversary of its incorporation on May 6. The firm has been a contributing member of ACI since 1919.

NRMCA Fellowships

The University of Maryland offers two Fellowships sponsored by the National Ready Mixed Concrete Assn., one designated the Stephen Stepanian Fellowship and one the C. Dolly Gray Fellowship. The Fellow enters upon his duties about September 1 or February 1 and continues for four consecutive semesters with a stipend of $1500 per year. Fees are paid and required textbooks are purchased by the Association.

Qualifications and study courses parallel those of the Stanton Walker Fellowship (p. 14) while the research work is devoted to problems appropriate to the ready mixed concrete industry. Forms for making application may be secured by writing to the Dean of the Graduate School, University of Maryland, College Park, Md.

James P. Michalos

The following corrections should be made in James P. Michalos' biographical sketch on p. 16 of the February 1952 News Letter.

Professor Michalos received his master's degree at Yale University and his PhD at Northwestern University.

Review materials controls

Representatives of the construction industry met in Washington February 12 to review the industry's situation under government controls. The conference was held by the Construction Industry Advisory Council, formed by the Chamber of Commerce of the United States. It is made up of more than 100 national trade and professional organizations with a major interest in construction. Defense Mobilizer Charles E. Wilson was principal speaker.

An afternoon session heard a panel discussion of the Controlled Materials Plan and the outlook for materials.

Ralph W. Kluge

Recently Ralph W. Kluge became head professor of civil engineering at the University of Florida, Gainesville. He was formerly associate professor of structural engineering at Purdue University.

Connors Steel rolling A 305 bars

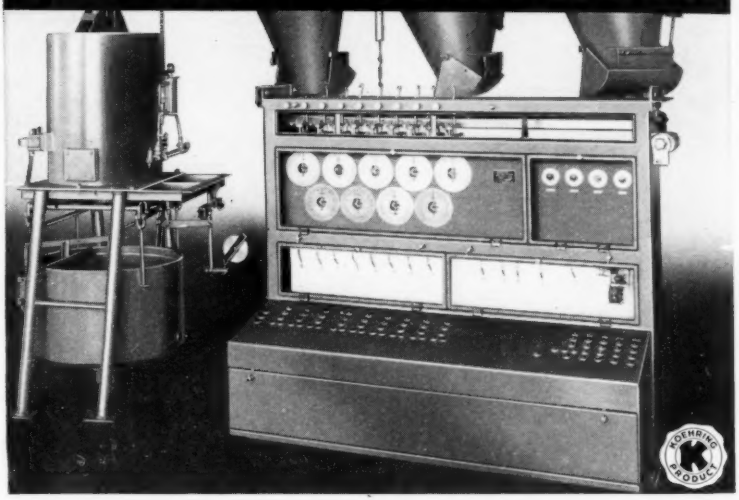
The Connors Steel Co., Birmingham, Ala., has announced that since January the company has been producing new deformed reinforcing bars meeting ASTM A 305 specifications.

Lehigh Portland Cement Co. establishes scholarship

An endowed undergraduate scholarship, open to any qualified student enrolled in the Engineering or Liberal Arts Departments, has been established at Princeton University by the Lehigh Portland Cement Co., Allentown, Pa.

Marking the first time in Princeton's history that an undergraduate scholarship has been endowed by a corporation, the newly created fund will be known as the Lehigh Portland Cement Company Scholarship and will return sufficient annual income to cover the full cost of an undergraduate's tuition for an academic year.

NERVE CENTER for RECORD PRODUCTION



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HIGH YARDAGE OUTPUT because . . .

Concentric zoning groups 1 to 5 mixers around center for most direct material flow, eliminates segregation of aggregates, and allows steep bottom slopes in aggregate hoppers for fast charging time. Central location of Cement Compartment permits steep slopes in cement tank for fast flow and maximum premixing action by introduction of cement into center of aggregate stream. Automatic Batcher Control features 2-step filling for fast and accurate weighing, instantaneous mix selection and fast automatic moisture compensation when moisture content is known.

QUALITY CONTROL because . . .

Separate single-material Batcher units operated from a master control unit (see photo above) are used for cement, water and each size aggregate. 2-step fill insures accurate weighing because intermittent feeding at close of charging operation reduces possibility of overload. Moisture compensator automatically corrects Water Batch scale to compensate for water content of sand. Adjustable retard on each Batcher gives optional delay in introducing Batch into mix.

ACCURATE PRODUCTION RECORDS because . . .

Chart pens operate directly from scale mechanism, record weight of each material on same sheet. Mixer consistency can also be shown. Electric counter records Batches discharged. Recorder also prints bars on chart paper, saving money and assuring accuracy of register.

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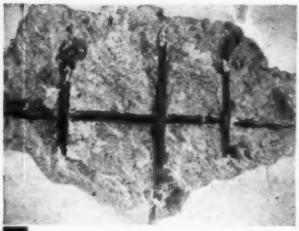
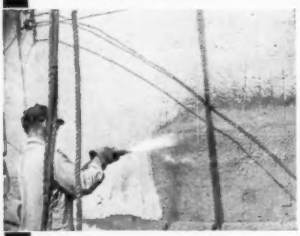


Typical example of spalling. Note corrosion of reinforcing rods exposed by disintegrating concrete.



Reinforced with meshing, the area is restored with gun-applied RESTO-CRETE* by Western Waterproofing Co.

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Ralph W. Hooper

Word was received at ACI headquarters recently of the death of Lieut. Ralph W. Hooper—a bomber squadron casualty. Before recall to active military service with the U. S. Army Air Force, Lieut. Hooper was employed by the Ring Engineering Co., general contractors, Washington, D. C., as construction engineer and acting superintendent. He joined ACI as a Junior Member in December 1951.

Birger Leon

Birger Leon, ACI member since 1944, died recently in Ottawa, Ontario, Mr. Birger was one of Canada's leading architectural engineers and chief of the projects division of the Department of Resources and Development. Until 1932 he was chief engineer with a steel firm in Toronto and was the designer of many Toronto buildings.

Walter P. Hirschberg

Walter P. Hirschberg, an architectural engineer who was an authority on reinforced concrete design, died recently in Milwaukee, Wis. He was president of the Federal Engineering Corp., specializing in design of industrial buildings. Mr. Hirschberg had been an ACI member since 1920.

Harry J. Kelly

Harry J. Kelly, manager of construction and maintenance for the Gulf Oil Corp., died in Pittsburgh, Pa., recently. Mr. Kelly was a civil engineering graduate of Texas A & M and had been an ACI member since 1950.

C. T. West

C. T. West, vice-president, purchasing agent and chief engineer of Monolith Portland Cement Co., Monolith, Calif., died recently. Mr. West had been affiliated with ACI since 1941 as a representative of his company.

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The new Veterans Administration Hospital, West Haven, Conn.

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Contractor: MERRITT, CHAPMAN & SCOTT

Darex AEA supplied by: CLARENCE WILEY, MILFORD, CONN.

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D A **DEWEY and ALMY Chemical Company**

Cambridge 40, Mass. — Chicago 38 — Montreal 32

Non-dusting, easy-to-clean "iron-clad" concrete floors that will wear four to six times longer than ordinary concrete floors were obtained in the Grace Street, Richmond, Va., parking garage by the use of Masterplate. It was embedded in the *surface* of the concrete while still in a plastic condition. Masterplate is a combination of specially processed, size-graded iron particles and Master Builders' cement dispersing agent. In addition to the advantages obtained in this parking garage, Masterplate produces non-colored or colored iron-armored floors which are spark resistant, static disseminating, corrosion resistant, non-slip and low cost. Millions of square feet of Masterplate iron-clad concrete floors are in use today and giving excellent service in such structures as — industrial plants, hospitals, warehouses, stores, service stations, ammunition depots, powder plants and similar fields.

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Inge Lyse

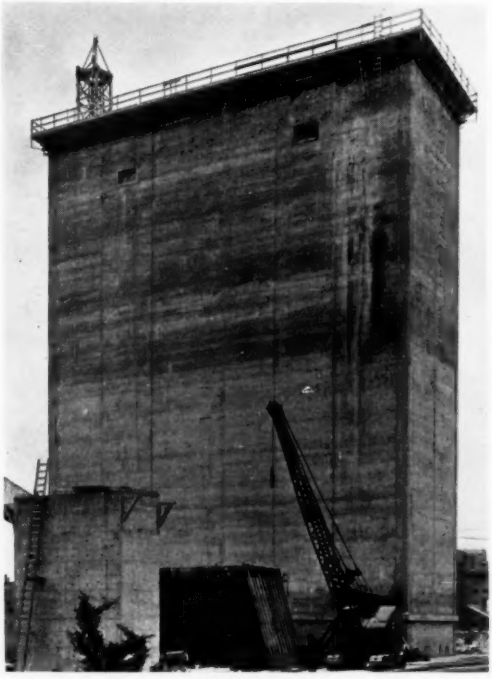
Inge Lyse is a name that will have familiar connotations for all Institute members and others whose memories go back to the inception of our Committee 115–Research, in 1936. Professor Lyse was its promoter, virtually its organizer and its first secretary.

A letter comes from Kharagpur, India, where Lyse writes (early January) from the Indian Institute of Technology where he is UNESCO representative. As Head of the Civil Engineering Department his mission is to set up this new Indian organization to serve along the lines of American technical institutions.

Kharagpur is 72 miles west of Calcutta and Lyse reports enjoying excellent weather at that time of year—80 degree F by day, 55 to 60 at night. He says "We hope it will stay that way for some time but old timers say in March it may really get hot."

"I am requesting the librarian to apply for ACI membership. *This* Institute is so very new periodicals of technical value have not yet been ordered. Soon we shall have a very fine technical library as well as a modern Institute with numerous well equipped laboratories, including a concrete laboratory."

—H. W.



Believed to be the tallest stone storage silo in the world, this 122-ft concrete structure was recently completed at the Maple Grove, Ohio, quarry of Basic Refractories, Inc. Housing 12 bins the silo will store approximately 10,250 tons of No. 6 (⅜-in.) stone in eight of the bins and, in addition, a substantial tonnage of supplemental raw material in the other four. Basic's No. 8 stone (passing 0.075 screen and meeting state specifications for concrete sand) was used in the concrete for this plant.

MacDonald Engineering Co. built the silo using sliding forms and worked round-the-clock under adverse weather conditions to complete it in a record eight days.

Errata

The following corrections should be made in "Concrete Footings for Walls and Columns," December 1951, JOURNAL: Expression in 10th line of section on "Bending moment in eccentrically loaded footing," p. 350 should read $w_m = q_m - q'$.

Eq. 50, p. 350 should read:

$$w = \left(q_m - \Delta q \frac{L}{0.5\,B}\right) - q'$$

$$= (q_m - q') - \Delta q \frac{L}{0.5\,B}$$

$$= w_m - \Delta q \frac{L}{0.5\,B} \quad \ldots\ldots\ldots\ldots\ldots (50)$$

Eq. 51, p. 351 should read:

$$BM = \frac{w_m L^2}{2} - \frac{\Delta q\, L}{0.5\,B} \times \frac{L}{2} \times \frac{L}{3}$$

$$= \frac{w_m L^2}{2}\left(1 - \frac{2}{3}\frac{\Delta q}{w_m} \times \frac{L}{B}\right)$$

$$= \frac{\omega\, w_m L^2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (51)$$

The following corrections should be made in "Direct Design of Rectangular Columns with Bending About an Axis of Symmetry," February 1952, JOURNAL:

Σ in "Notation," p. 467, should read ϵ

Thirteenth line under "Balanced Design of Sections with Symmetrical Reinforcement," p. 475 should read: *i.e.*, $d' = d''$. Substituting for M_2 from Eq. 20 and ...

Twelfth line, p. 478, should refer to Fig. 14.

Fifth line under "Conclusion and Summary," p. 482 should read:

with a moment $M = N\ (\epsilon + hd)$, where $\epsilon + hd$ is the ...

Line 24, Appendix 1, p. 484, should read:

$$-\frac{6N\epsilon/d}{F_c bd}(k^2 - g) + \frac{3N}{F_c bd}(1-g)(g-k^2) + \frac{3N}{F_c bd}(k-g)^2$$

Stanton Walker Fellowship

Established in honor of Stanton Walker, long known for his work in the concrete field, the University of Maryland offers the Stanton Walker Fellowship sponsored by the National Sand and Gravel Assn. The Fellow enters upon his duties about September 1 or February 1 and continues for four semesters. The stipend is $1500 per year paid in ten monthly installments. Fees are paid and required textbooks are purchased by the Association.

The appointee to the Fellowship will register in the Graduate School of the University of Maryland. Work will be scheduled so that the Fellow's time will be divided about equally between study of selected and approved courses and research on appropriate problems related to the sand and gravel industry.

Completion of the work leads to the degree of Master of Science. The Fellowship is open to qualified graduates in engineering from accredited colleges and universities. Forms for making application may be secured by writing to the Dean of the Graduate School, University of Maryland, College Park, Md.

CAN YOU SUPPLY CORRECT ADDRESSES?

Following is a list of members of the Institute (with their last known addresses) with whom we are out of touch, mail being returned for one reason or another. If any members of the Institute reading this paragraph could help us with the present correct addresses of the members noted, it would be very much appreciated:

A. D. ALLEN, JR., P. O. Box 371, Jacksonville 1, Fla.
JIM BERRY, c/o Behfontain Apts., Apt. 2, 3310 Moroneal, Houston, Texas
ILYAS ELFIYE BINLER, Box 543, W. Lafayette, Ind.
GORDON L. BURT, 5001 N. Columbia Blvd., Portland 3, Ore.
ALVARO CABAL, 839 Convention, Baton Rouge, La.
GUSTAVO CACERES, Calle 13 No. 14-12, Bogota, Colombia
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JOHNNY D. GIMARČ, Box 5451, College Station, Texas
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RUSTOM ARDESHIR IRANI, c/o Apt. 4A, 146 W. 49th St., New York 26, N. Y.
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WALTER S. MACLAUGHLIN, 30 Wallace Rd., Binghampton, N. Y.
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LUTFULLAH MALKOCOGLU, 13558 223rd St., Laurelton, L. I., N. Y.
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ROBERT M. SCHROLL, Kappa Sigma House, Laramie, Wyo.
ROBERT GEORGE SCHWENDLER, 128 Walsh Hall, Notre Dame, Ind.
FRANK L. SPILLERS, P. O. Box 641, Denver, Colo.
JOSE M. TAMAYO, 5949 Park Pl., Hammond, Ind.
FRANCISCO JOSE TANG, P. O. Box 236, Angola, Ind.
KWANG-YUEN TAO, South Quad 217, Iowa City, Ia.
DANTE VENTRIERI E., Pasaje E. No. 71 apto 1, Buenavista, Marianao, Cuba
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Individual

ACKERMAN, HARRY, Inginiyagala, Ceylon (Office Engr', Morrison-Knudsen International Co., Inc.)
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ALLEN, M. G., Providence, R. I. (Chf. Engr., Gilbane Building Co.)
BENSON, ROBERT, Farnborough, Kent, England (Asst. Struct. Engr., Ministry of Works)
BISSONNETTE, WILLIAM A., McNary, Ore. (Super. of Inspection and Control, Corps of Engrs.)
BLANCH G., OSVALDO J., Caracas, Venezuela (Cons. Engr.)
BOLE, EUGENE M., Shaker Heights, Ohio (Asst. C. E. Cleveland Electric Illuminating Co.)
BOURGEOT, GILBERTO, Caracas, Venezuela (Design, Oficina Tecnica de Ingenieria)
CARPENTER, A. R., San Mateo, Calif. (Estimating Job Super., Parker, Steffens & Pearce)
CASTRO-CORDOBA, JAIME, Cambridge, Mass. (Constr.)
COBLANS, HERBERT, Durban, S. Africa (Univ. Librarian, Univ. of Natal)
CRAWFORD, C. S., Philadelphia, Pa. (Exec. Vice Pres., The Whitehall Cement Mfg. Co.)
DAHL, ROBERT E., Manhattan, Kans. (Research Asst., Kansas State College)
DEEMER, RAY H., Huntington, W. Va. (Sale Engrg., The Master Builders Co.)
DOCTOR, P. K., Karachi, W. Pakistan (Dist. Engr., The Concrete Assn. of India)
DUARTE, EDUARDO GARCEZ, Parana, Brazil (Partner, Sociedade Tecnica Ltd.)
EDSON, MURRAY H., Roselle, N. J. (Tech. Editor, Lefax, Inc.)
FEHNEL, C. D., Hudson, N. Y. (Chf. Chem., Lone Star Cement Corp.)
FERNANDEZ, FELIPE GORBEA, Santurce, P. R. (Struct. Engr., U. S. Navy)
FIELD, CYRUS E., Minneapolis, Minn. (Genl. Constr., C. D. Field Co.)
GEDENO, DELFIN, Caracas, Venezuela (Design, Inos, Edificio "Las Mercedes")
GERHARDT, NORMAN A., Los Angeles, Calif. (Struct. Design, Drafting, Henry M. Layne, Engr.)
GINGERICH, HAROLD R., Corpus Christi, Texas (Chf. Chem., Halliburton Portland Cement Co.)
HAAS, WILLIAM M., Pittsburgh, Pa. (Drafting, Layout, Design, Walter H. Frick, Engr.)
HAMILTON, JOHN S., Chicago, Ill. (Sales Engr., E. W. Zimmerman)
HANSON, HAROLD E., Duluth, Minn. (Design, Super., Harold S. Starin, Arch.)
HETTERICH, RALPH H., Hamilton, Ohio (Partner, Mueller, Hair & Hetterich)
HUNT, J. T., London, England (Tech. Repr., Dewey & Almy Ltd.)
IDEN, HARRY C., Mechanicville, N. Y. (Chf. Draftsman, W. Va. Pulp & Paper Co.)
IWANAGA, GEORGE SHINICHI, Los Angeles, Calif. (Design Engr., Los Angeles County Road Dept.)
JIMENEZ, FELIPE, Champaign, Ill.
JORGENSEN, IB FALK, Englewood, Colo. (Civil & Struct. Engr.)
KENNEDY, DOUGLASS J., Holyoke, Mass. (Design, Super., P. J. Kennedy & Co.)
KIRKHAM, R. E., Omaha, Nebr. (Engr., Kirkham, Michael & Assocs.)
KRASNER, LOUIS, University City, Mo. (Struct. Engr.)
LAMACK, AUGUST H., St. Louis, Mo. (Design, Promotion, Inspection, Laclede Steel Co.)
LAUGA, HENDRIK, Toronto, Canada (Struct. Engr., Canadian-Brazilian Services, Ltd.)

Continued on p. 17

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Continued from p. 15

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LAYA MORALES, PEDRO, Caracas, Venezuela (Design, Inspection, Instituto Nacional de Obras Sanitarias)
LEE, DAH HSUAN, New York, N. Y. (Design, J. G. White Engrg. Corp.)
LUKER, J. P., Houston, Texas (Research Lab. Manager, General Portland Cement Co.)
MAUGEE, J. R., Paris, France (Sales Engr., Dewey & Almy Chemical Co.)
McMANUS, RALPH N., Edmonton, Canada (Asst. Prof., Univ. of Alberta)
McPHERSON, RALPH H., Greenville, S. C. (Partner, Chf. Engr., Chf. Arch., The McPherson Co.)
MITCHELL, WARRINGTON GILLET, Fresno, Calif. (Chf. Concrete Tech., Corps of Engrs.)
MITTET, H. P., Seattle, Wash. (Asst. Prof., Univ. of Washington)
MORENO, ALBERTO JOSE, Caracas, Venezuela (Design, Instituto Nacional Obras Sanitarias)
NELSON, DELLIEWARE R., Cambridge, Mass. (Lab. Testing, Field Super., Inspection, H. G. Protze, Engr.)
NEWMAN, ALBERT A., Hackensack, N. J. (Design, Drafting, Jacoby, McGrayne & Co.)
NORCROSS, L. T., Snyder, Texas (Genl. Bldg. Contr., Norcross Constr. Co.)
NORRIE, KENNETH PETER, Spokane, Wash. (Struct. Design, Struct. & Civil Engr.)
O'BRIEN, DANIEL E., Crystal Lake, Ill. (Jr. Engr., A. J. Boynton, Inc.)
PALERMO, WILLIAM LYNN, Los Angeles, Calif. (Estimator, J. A. McNeil Co., Inc.)
PARMA, JOHN W., Cleveland, Ohio (Design, Drafting, Wilbur Watson Assocs.)
PINO, DIOGENES ALBERTO, Panama, Panama (Design, C. E.)
QUINBY, IRA, Kenmore, N. Y. (Struct. Field Engr., Portland Cement Assn.)
RAYBURN, E. B., Jr., Indianapolis, Ind. (Asst. Genl. Mgr., Ready Mixed Concrete Corp.)
ROSENSTERN, KLAUS, Chicago, Ill. (Squad Leader, Freyn Engr. Dept. of Koppers Co.)
RUSSELL, WILLIAM N., Jr., Westmont, N. J. (Genl. Mgr., W. N. Russell & Co.)
SHAW, ARTHUR JOSEPHUS, Jr., Quincy, Mass. (Design, Drafting, Mass. Institute of Technology)
SOLOMON, JOSEPH S., Los Angeles, Calif. (Struct. Design, The Austin Co.)
SORENSEN, WENDELL C., Shreveport, La. (Chf. Engr., Assoc. Struct. Design, Neild-Somdal-Assocs.)
STEYN, KEEVE, Johannesburg, S. Africa (Cons. Engr.)
SWATEK, EDWIN PAUL, Jr., Pittsburgh, Pa. (Chf. Design Engr., Dravo Corp.)
TRUEN, EDWARD K., Concord, Mass. (Arch., Struct. Engr.; Instructor, Harvard Univ.)
UCHIYAMA, MINOYU, Tokyo, Japan (Prof., Chuo University)
VAMOS, LADISLAO, Caracas, Venezuela (Genl. Mgr., Weil y Vamos)
VASARHELYI, D., Seattle, Wash. (Research, Univ. of Washington)
WATSON, L. T., Jr., Chicago, Ill. (Struct. Design, U. S. Navy)
WELBORN, MORRIS F., Hayward, Calif. (Design, Estimating, International Engrg. Co., Inc.; Morrison-Knudsen International Co., Inc.)
WILLSON, CEDRIC, Dallas, Texas (Vice-Pres., Chf. Engr., Texas Industries, Inc.)
WOLFE, G. MORTON, Buffalo, N. Y. (Arch., Engr.)

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CHIBA, SHIZUO, Hokkaido, Japan (Asst., Dept of Bldg. Constr., Hokkaido Univ.)
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CLARKE, DENNIS, Newcastle-upon-Tyne, England (Design, Draftsman, The British Reinforced Concrete Engrg. Co., Ltd.)

Continued on p. 21

Honor Roll

February 1, 1951—January 31, 1952

For the final tally of this year's Honor Roll, Newlin D. Morgan, Sr., leads with 40 credits, Newlin D. Morgan, Jr., is second with 17½, Eddy Hernandez is third with 10½ and James A. McCarthy is fourth with 10 credits.

Each of the Morgans for the fourth consecutive year will receive a special certificate of appreciation authorized by the Board of Direction.

The entire list is slightly over 8 percent of the total membership helping others derive the benefits that ACI offers. Why should this not be 10, 15 or even 20 percent?

Newlin D. Morgan, Sr. (Ill.)	40
Newlin D. Morgan, Jr. (Mich.)	17½
Eddy Hernandez (La.)	10½
James A. McCarthy (Ind.)	10
J. E. Jellick (Calif.)	9
Oscar A. Nunez G. (Venezuela)	7½
Adolf A. Meyer (Tenn.)	7
Jose A. Vila (Cuba)	7
H. J. Gilkey (Ia.)	6½
L. G. Farrant (Fla.)	6
Samuel Hobbs (Calif.)	6
Alvin C. Loewer, Jr. (Pa.)	6
Jacob Adler (Israel)	5½
F. Thomas Collins (Calif.)	5½
Anthony G. Giardina (N. Y.)	5
Frank Kerekes (Ia.)	5
Alexander Klein (Calif.)	5
C. P. Siess (Ill.)	5
C. H. Scholer (Kans.)	4½
Oscar Schreier (N. Y.)	4½
Oliver J. Julian (Mass.)	4
Henry L. Kennedy (Mass.)	4
Fernan Rodriguez-Gil (Venezuela)	4
Andrew Verkade (Ill.)	4
Ivan M. Viest (Ill.)	4
George D. Youngclaus (Calif.)	4
George E. Hatch (Hawaii)	3½
Blas Lamberti (Venezuela)	3½
F. N. Menefee (Mich.)	3½
Rafael Ruiz (Guatemala)	3½
R. A. Caughey (Ia.)	3
Oscar A. T. Gimesy (Australia)	3
I. E. Morris (Ga.)	3
Howard Simpson (Mass.)	3
LeRoy A. Thorssen (Canada)	3
Curzon Dobell (N. Y.)	2½
Myle J. Holley, Jr. (Mass.)	2½
E. L. Howard (Calif.)	2½
T. C. Kavanagh (Pa.)	2½
William Lerch (Ill.)	2½
F. T. Mavis (Pa.)	2½
Donald Agrimson (S. D.)	2
Boyd G. Anderson (N. Y.)	2
Michel Bakhoum (Egypt)	2
J. F. Baxter (N. Y.)	2
Gene Best (Texas)	2
E. M. Bowles (Ceylon)	2

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Stanley L. Howell (Nebr.) 1

Continued on p. 20

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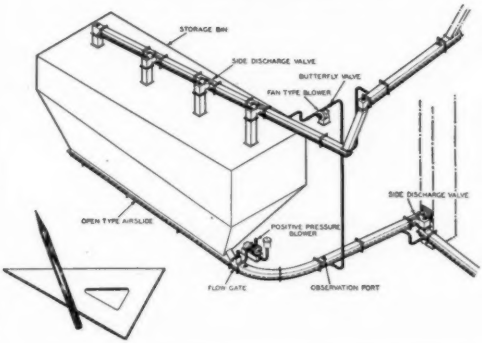

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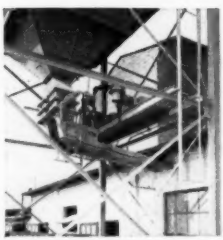
Conveying from elevator to storage bin underneath floor

Open-type Airslide in bin bottom conveying to elevator



Conveying from storage bin to weigh scales in plant

Honor Roll

Continued from p. 18

S. E. Huey (La.) 1
Robert B. Hyslop (Wash.) 1
Leslie A. Irwin (Calif.) 1
Gordon Jacoby (N. J.) 1
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Matti Janhunen (Finland) 1
Robert B. Jenkinson (Calif.) 1
J. G. Johnson (N. Y.) 1
W. R. Johnson (Ore.) 1
Barton Jones (P. R.) 1
R. R. Kaufman (Ohio) 1
Thomas M. Kelly (Ore.) 1
K. J. Kelsall (Australia) 1
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Antoni Martynowicz (Canada) 1
Bryant Mather (Miss.) 1
D. M. McCain (Miss.) 1
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William G. McFarland (P. R.) 1
Douglas McHenry (Colo.) 1
Ralph H. McPherson (S. C.) 1
B. S. Merrill (Tenn.) 1
Gennaro Mianulli (N. Y.) 1
T. C. Miller (Ill.) 1
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Charles C. Zollman

New Members

Continued from p. 17

FARIS, CHARLES O., Mill City, Ore. (Constr. Engr., Consolidated Builders, Inc.)
GALLANGO M., LUIS NAPOLEON, Caracas, Venezuela (C. E., Tecnica Constructora)
GARDON, JUAN S., Catano, P. R. (Genl. Constr., Inspection, P. R. Housing Authority)
GILBERT, CHARLES WADE, Pampa, Texas (Struct. Design, Cabot Carbon Co.)
HIGGINS, FRANK T., Jr., Woodstown, N. J. (Lab. Testing, Inspection, J. E. Greiner Co.)
KOLOZENSKI, EDMUND A., Franklin Park, Ill. (Struct. Design, A. J. Boynton, Inc.)
LONG, CARL F., Cambridge, Mass. (Teaching Asst., Mass. Institute of Technology)
MANFULL, DUANE, New York, N. Y. (Lt., U. S. Army)
MINCKLER, HOWARD G., Bridgeport, N. J. (Inspector, Lab. Testing, J. E. Greiner Co.)
MLAGENOVICH, WILLIAM M., Whittier, Calif. (Engrg. Design, Drafting, Schrieber & Peterson)
MUNIZ, ARTURO ORTIZ, Guayama, P. R. (Constr. Engr., Nieves Lumber & Constr. Corp.)
NASR, MAHMOUD SAYED ALY, Giza, Egypt (Instructor, Fouad 1st University)
NESTER, JAMES D., Middletown, Ia. (Design, Silas Mason Co.)
PATON, ARCHIE, Oak Park, Ill. (Design, Drafting, Inspection, A. J. Boynton & Co.)
REIBER, MARC BRUCE, Laurelton, N. Y. (Design, Albert Goldberg, Engr.)
RICE, CLAIR E., Mandon, N. D. (Constr., Design, N. D. State Highway Dept.)
RIZK, AMIR ASSAD, Giza, Egypt (Instructor, Fouad 1st University)
ROBINSON, RHEES RICHARD, Jr., Kansas City, Mo. (Jr. Engr., Howard, Needles, Tammen & Bergendoff)
ROMMEL, RICHARD E., Chicago, Ill. (Struct. Design, Drafting, William Schmidt, Engr.)
SACHS, S. L., Montreal, Canada (Struct. Design, Dominion Struct. Steel Ltd.)
SORENSON, S. E., Corpus Christi, Texas (Struct. Draftsman, Blucher & Naismith, Inc.)
STEWART, M. W., Snyder, Texas (Engr., R. F. Ball Constr. Co.)
YAMAMOTO, Y. Eugene, Chicago, Ill. (Design, A. J. Boynton, Inc.)

Student

CANO, VICARIO RUBEN, Mexico, Mexico
CHIN, TIMOTHY K., Ames, Ia. (Ia. State College)
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GIRTON, DARREL D., Ames, Ia. (Ia. State College)
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MARQUEZ, J. MIGUEL, Pittsburgh, Pa. (Carnegie Inst. of Tech.)
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ROY, LEO, Jr., Lafayette, Ind. (Purdue Univ.)
SELL, ROBERT F., Madison, Wis. (Univ. of Wis.)
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YUAN, SHANG WEN, Berkeley, Calif. (Univ. of Calif.)

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Air-operated mounting clamp

An air-operated mounting clamp has been designed to speed up attachment of external vibrators to load. It is claimed to be extremely advantageous when short periods of vibration are required. It is reported that no lengthy installation is necessary as air pressure applied first to the clamp causes specially designed jaws to grip securely onto a suitable steel member such as an exposed angle.

It is said to be extremely beneficial in unloading trucks where in the past, lengthy time required in attachment of the vibrator to the load made vibration impracticable. Additional information may be obtained upon request.—Viber Co., 726 S. Flower, Burbank, Calif.

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DISCUSSION

Discussion closed January 1, 1952

Sept. Jl. '51

Earthquake Resistant Design Considerations—R. R. Martel

Comprehensive Numerical Method for the Analysis of Earthquake Resistant Structures—Charles S. Whitney, Boyd G. Anderson and Mario G. Salvadori

Multistory Buildings Designed to Resist Earthquakes—John J. Gould

Lightweight Concrete for Lower Construction Costs—J. A. Murlin

Designing for Continuity in Prestressed Concrete Structures—Alfred L. Parme and George H. Paris

Pumice—Lightweight Aggregate—Leslie I. Neher

Hydration Products Formed in Cement Pastes at 25 to 175 C—George L. Kalousek and Milton Adams

Discussion closed February 1, 1952

Oct. Jl. '51

Advances in Precast Floor Systems—F. N. Menefee

Manufacture and Use of Machine-Made Precast Structural Elements—A. G. Streblow

Tilt-up Construction in Western United States—F. Thomas Collins

Diagonal Tension in Reinforced Concrete Beams—Arthur P. Clark

Coral and Salt Water as Concrete Materials—John G. Dempsey

Load Carrying Capacity of Dowels at Transverse Pavement Joints—Henri Marcus

Exciting Adventures*

By HARRY F. THOMSON†

SYNOPSIS

Retiring ACI President Thomson reviews the year's activities of the Institute. He considers membership growth, financial condition, staff activities, committee work, special publications, intersociety relations and Institute meetings. He looks forward to increasing development of new techniques in concrete and new applications of this versatile material.

OBJECTIVES

The justification and the objective of the American Concrete Institute are expressed pointedly in the by-line at the bottom of the cover of our JOURNAL. This line reads—"to provide a comradeship in finding the best ways to do concrete work of all kinds and in spreading that knowledge." The comradeship envisioned in this statement is evidenced by the innumerable contacts made at our annual and regional meetings, and by extensive unofficial correspondence resulting from these personal contacts. We search for the best ways to do concrete work of all kinds through the steady stream of research and design work carried out by individuals and groups, and then summarized in papers and committee reports. And finally that knowledge is spread through the publication of these papers and reports in the Institute's JOURNAL or in special publications distributed to our membership and others who are interested. The Institute staff itself does not conduct the research and development work, but serves as the clearing house through which progressive ideas in all fields of the utilization of concrete are passed on to the public.

MEMBERSHIP

This function of serving as a clearing house is a sacred mission, even though self-assumed. If we serve well in the nation's economy, our organization grows; if not, we tend to wither. I am happy to report that by the two most tangible measures the American Concrete Institute is currently at the top of its long career. In point of members, we passed the level of 5500 early last month which was a sight set some years ago when your officers visualized doubling the membership. This happy result did not come from any sporadic membership drive but from a steady accretion which has testified to the merit of the Institute's service.

*Presented at the ACI 48th Annual Convention, Cincinnati, Ohio, Feb. 27, 1952. Title No. 48-39 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 8, Apr. 1952, *Proceedings* V. 48. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Aug. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Consulting Engineer, St. Louis 5, Mo.

FINANCES

Likewise in finances, our last year's budget was the largest in our history, and I am especially happy that our year's operations resulted in a comfortable balance in the black. The increase in dues rates which was ordered to letter ballot at our business meeting a year ago, and subsequently approved to become effective last July 1, of course contributed to this favorable showing. Your officers have appreciated the expression of confidence by our members in approving this increase. At the same time, the staff are to be commended for exercising economies in the face of advancing unit costs, without curtailing services. For 1952 our estimated income of $155,000 will slightly exceed that of 1951 and should enable the Institute to resume the goal set by your directors some years ago of allotting a small percentage of gross income to reserve. It is interesting to note that dues account for only 55 percent of our gross income, most of the balance coming from sale of publications and advertising.

LITERARY CONTRIBUTIONS

Like other voluntary associations, our Institute depends on the mental and literary contributions of its members for the material presented in papers and reports, which reflect progress in our field of utilizing concrete. Authors of papers receive a certain amount of acclaim when their papers are published. The staff are continually on the lookout for more short papers, those which reflect new and adventurous thinking. A paper should not be withheld because it is not, in its original form, a literary gem; submission of the paper is invited, with the expectation that the editors will do the required polishing, but without destroying the impression created by the author. A greater exchange of ideas will aid the value and virility of our JOURNAL.

COMMITTEE WORK

But the unheralded workers for the Institute are the committee members; to these men I want to express a word of gratitude on behalf of our entire membership. There are 265 men serving on our working technical committees. To me it has been an inspiration to note the continuous flow of correspondence among these members. Twenty-one meetings of committees were held during this convention. In addition, several committees have held meetings during the year at other times than the Institute's general meetings, attended in many cases by men who had no financial interest in the problems under discussion but whose attendance represented personal sacrifices. When committee reports appear, we should recognize the serious deliberations which they summarize. To our committee members I extend a sincere "Thank You."

SPECIAL PUBLICATIONS

A notable addition to our list of special publications, which it is expected will be issued shortly, is the "Proposed Manual of Standard Practice for Detailing Reinforced Concrete Highway Structures." This document,

together with the original "Manual of Standard Practice for Detailing Reinforced Concrete Structures," which has been revised and reprinted to incorporate the 1951 Building Code changes resulting from adoption of the A305 bar, represents a great contribution to the reinforced concrete field. These manuals are destined to become the standards for detailing practices. Great credit is due the members of the joint committee of ACI and the Concrete Reinforcing Steel Institute for the painstaking work during almost 10 years in developing these most-important contributions.

But while bringing out these new manuals the Institute is not neglecting the task of keeping our other widely-used special publications in line with modern practices. Our Building Code Committee is working on the modernization of the Code. This document is either covered by reference or forms the basis for provisions for concrete and reinforced concrete sections in many of the new building codes adopted in recent years. Revisions of our Inspection Manual and our Concrete Primer are also under way.

INTERSOCIETY ACTIVITIES

In the realm of contacts with other technical groups, our relations continue most cordial. Our steel friends were very cooperative in publicizing the Code provisions for the new A305 bars, which were approved by our convention a year ago. We were happy to be one of the six sponsors for the First Conference on Prestressing held at Massachusetts Institute of Technology last summer which, by the way, designated the ACI as the organization to codify terminology and procedure in connection with prestressing. We were actively represented at the International Concrete Conference at Antwerp and at the Building Conference in London last summer. Our regional meeting next September will be held in Chicago at the time of the Convocation of Engineers which was initiated by the American Society of Civil Engineers. This will be a joint meeting with the Structural Division of ASCE, for which our committee has already arranged a full two-day program of important papers, including two by authors from outside this country. The project of producing instructional motion pictures of which we are co-sponsors with the Portland Cement Assn., is proceeding under a joint advisory committee of PCA and ACI. It is expected that two films on quality concrete will be completed before the end of 1952. Such cooperative activities form an important segment of ACI's functioning.

Another important activity of the past year was the regional meeting in St. Louis in October. This gathering was highly successful, both in attendance and in interest. Like our previous fall meetings, all arrangements, including the program, were handled by a local committee. Several innovations in the way of entertainment added a new zest to an Institute gathering.

STAFF CHANGES

With the close of this convention, our organization passes an important milepost, or rather a 32nd milepost, as that is the length of service by Harvey

Whipple as our Secretary and subsequently Secretary-Treasurer. To many of our members and friends his personality has epitomized the energy, the judgment and the foresight of the Institute. He has earned the title among this generation of "Mr. ACI." His retirement will remove a unique atmosphere from the daily correspondence which many of us will miss. But all who have known Mr. Whipple will wish him contentment after he drops the reins of active office, and we trust we shall have the pleasure of greeting him at our conventions for many years to come.

Your Board of Direction is confident that the affairs of the Institute will continue to be carried on effectively. Our headquarters staff is small but well trained. And Fred F. Van Atta, who has been designated Acting Secretary-Treasurer, after six years experience on the staff, is well equipped to continue the service to which our members are accustomed.

FUTURE PROSPECTS

Although we like to think of concrete as a universally accepted building material, I am convinced that we can look forward to new adventures in its applications. One cannot predict today what will be the developments of tomorrow, but progress in both design and construction is sure, and some of the results will bring excitement. I like to think of the story told of M. Freyssinet, the French structural designer, to the effect that after placement of concrete in one of his important arch bridges his excitement was such that he refused to wait the conventional 28 days, or even 7 days, but ordered the centering removed after only 3 days, just to see if the bridge would stand—and it did! Similarly I think we can look for important advances in some of the concepts which have come forward in recent years. These include prestressing and thin shells, both of which must prove their economic values under the conditions of material and labor costs in this country. There are also less spectacular fields such as precast units, structural shapes, light weight aggregates, and various methods of small house construction.

Cement paste can be considered as a type of plastic; who can say that some day there will not be available an additive which will cause concrete to develop an important tensile strength, and thereby create a new phase of structural practice?

The forerunner of these and other advances is the research work carried on in the large number of public and private agencies studying concrete. We will agree with the author of a recent article on concrete in a popular magazine who stated: "A healthy industrial technology keeps its basic research far enough ahead of practice to encourage constant improvement in that practice."

We should not sell concrete short. It is a wonderfully versatile material, not yet developed or utilized to its full extent. If we think of it as "plastic stone," we realize it is subject to our manipulation. And some of the results of that manipulation will bring exciting adventures on the road of progress. The prime objective of the Institute is to aid that progress.

Title No. 48-40

Wave Velocity in Concrete*

By JOHANNES ANDERSEN† and POUL NERENST‡

SYNOPSIS

Nondestructive testing of concrete has aroused increasing interest throughout the world as it presents a new approach to determination of concrete quality. A Danish timing device and its application to determination of the wave velocity in concrete specimens is described. The progress of hardening of concrete specimens was followed, and a hypothesis for the relation between wave velocity and age is presented. The method has been used on concrete *in situ* including measurements of concrete members damaged by exposure to fire or freezing at an early age. Measurements on concrete pavements give wave velocities with small variation. An appendix presents detailed procedure for calculating wave velocity and an estimate of concrete homogeneity.

INTRODUCTION

Control of concrete quality has until recently been limited to destructive testing methods, and in most cases the testing has been carried out on specimens. By destructive methods only a single test result is obtained from each specimen, and test series are accordingly very comprehensive as the scatter of results often approaches the differences expected between different series of test specimens.

Nondestructive testing makes it possible to obtain many test results from a single specimen and thus permits following changes in the properties of concrete with time and external influences. Thus the resonant frequency method for the determination of dynamic modulus of elasticity developed in U. S. has been of great importance to the study of deterioration of concrete due to freezing and thawing cycles.

The wave velocity method is also nondestructive but has a wider applicability than the resonant frequency method because the method may be applied to actual structures.

Different apparatus and measuring procedures have been developed in the United States,[1] Canada,[2] England[3] and France.[4] In Denmark a wave velocity testing method,[5] has been developed and results were described in a report by the Danish National Institute of Building Research.[6] This paper is based upon these publications and later experience with the method. After a description of the apparatus and the measuring technique the varia-

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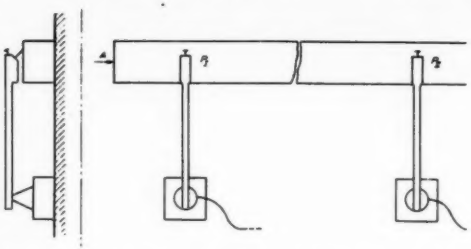

Fig. 1—Placing of pick-ups for determining longitudinal wave velocity of beam (*Foss Brink and Christensen-Dalsgaard*[5])

tion in wave velocity of concrete specimens with curing conditions and age is considered. The latter part of the paper deals with measurements on actual structures.

APPARATUS AND PROCEDURE

The condenser chronograph designed by Danish engineers Joergensen and Weibel has been used for ballistic measurements. An apparatus of this type also was used for the preliminary investigations at the Laboratory of Acoustics of the Academy of Technical Science. The following description is partly based upon a paper in Danish by Brink and Christensen-Dalsgaard.[5]

Fig. 1 illustrates the location of pick-ups for measuring propagation velocity of longitudinal waves in a beam. The pulse is produced in the beam by an axial blow against one end, A, of the beam. The waves produced are received by two pick-ups P_I and P_{II}.

Crystal pick-ups seem to be superior to those of the magneto electric type. Experiments have shown that satisfactory contact is obtained when the pick-up needles simply rest on the surface of the object tested. Mechanical and electrical properties of the two pick-ups need not be exactly equal when several different pick-up spacings are used.

Time measuring device

The principle of the condenser chronograph is shown in Fig. 2. B is a battery, the terminals of which are connected in parallel with a resistance, R, a condenser, C, and a potentiometer with a resistance of r_1. RV is a calibrated dc vacuum-tube voltmeter. I and II are relays governing the electric connection to contact a and contacts a' and b', respectively. Initially, where

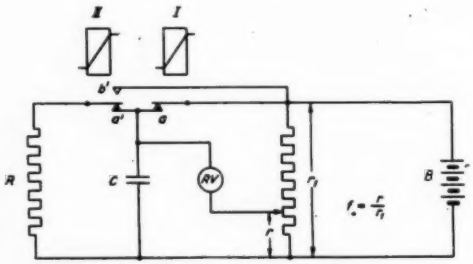

Fig. 2—Schematic diagram of condenser chronograph (*Foss Brink and Christensen-Dalsgaard*[5])

contacts a and a' are connected, condenser C is charged up to a certain voltage V_0. The chronograph is suitable for recording the time-interval between two electrical impulses, these impulses being brought to close each of the two direct current circuits. The direct current in the circuit closed by the first impulse passes through the coil of relay I, whereby contact a is disconnected. Thereby a discharge of condenser C through the resistance R is initiated.

The direct current in the circuit closed by the second impulse passes through the coil of relay II, whereby contact a' is disconnected and contact b' is connected. This interrupts the discharging of condenser C, the voltage of which at this time has decreased to a value V_1. The time interval, t_1, between the disconnecting of contacts a and a' is found from:

$$V_1 = V_0 \times e^{-\frac{t_1}{R_1 C_1}} \quad \ldots\ldots (1)$$

Where

e = the base of natural logarithms
R_1 = resistance of R
C_1 = capacity of C

Then

$$t_1 = -R_1 C_1 \log_e \frac{V_1}{V_0} \quad \ldots\ldots (2)$$

When R_1, C_1 and V_1/V_0, the ratio of the condenser voltages before and after the discharge are known, t_1 can be found. After the measuring period, the contact b' is connected, so that the direct current load, and, consequently, the voltage at the terminals of battery B is the same as immediately before the measuring period.

The V_1/V_0 ratio may, therefore, be found by adjusting potentiometer r_1 until the tube-voltmeter indicates zero voltage difference at its terminals. Suppose now the voltage-ratio of the potentiometer to be f, then

$$f = V_1/V_0 \quad \ldots\ldots (3)$$

and consequently

$$t_1 = R_1 C_1 \log_e f \quad \ldots\ldots (4)$$

In practice the potentiometer r is provided with decades of tenths and, prior to the measuring period, the vacuum-tube voltmeter is so calibrated that full deflection corresponds to 1/10 of the voltage V_0. The ratio f is thus determined to three decimals through combination of the reading on the potentiometer and the reading on the tube voltmeter RV.

In making a measurement the two pick-ups P_I and P_{II} (Fig. 1) govern the direct current circuits through the coils of relay I and II, respectively (Fig. 2). The principle of the governor circuits of the relays will be apparent from Fig. 3.

The pick-up voltage passes, screened, through a stepwise adjustable HP filter of the RC type. The filter is introduced to damp disturbing low-frequency impulses originating from possible irrelevant vibrations of the pick-up; the frequency limit is 500 rps. From the filter the pick-up voltage is

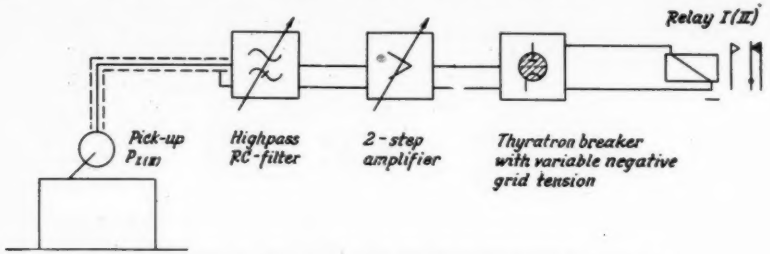

Fig. 3—Schematic diagram of relay circuits of condenser chronograph

carried to the primary potentiometer of a two-step resistance coupled amplifier, and further to a thyratron with negative grid tension, and the coil of relay *I* (or *II*) is inserted in the anode circuit of the thyratron (see Fig. 3). When the amplified pick-up voltage led to the thyratron grid exceeds a certain value the thyratron will let the current pass through the coil of relay *I* (*II*), whereby the relay comes into action.

Generally a sensitivity of 1 mv is applicable whereby a slight blow on the object, for example with a fingernail, near the pick-ups will actuate the thyratrons.

As mentioned above the two pick-ups need not possess exactly equal properties. Correspondingly the two relays, *I* and *II* in Fig. 2, and their governor circuits (Fig. 3) need not be completely equal. Linear differences of the two pick-ups, the governor circuits, or the relays will appear as a constant error, t_0, in the time, t_1, recorded. t_0 may be termed zero-time and may be ascertained when the pick-ups are so placed that they are simultaneously actuated by the wave pulse. t_0 can be read directly on the chronograph, if only the greater pulse delay is found in the circuit to relay *II*. This has been taken into consideration in the design of the apparatus. It may be necessary to connect the pick-up with the greater delay to relay circuit *II* to obtain a positive, and measurable, value of t_0. The zero time, t_0, can be changed through an adjustment of the amplification of the relay circuit. When the propagation time of the wave pulses in an elastic material is measured as a function of the pick-up spacing, both propagation time and zero-time can be determined graphically or by computation.

Measuring technique

In the preliminary investigations at the Laboratory of Acoustics the test specimens were marked at 10 cm intervals, the pick-ups being placed at these marks. The wave pulse was produced by an axial blow with a hammer, either directly by hand or by letting the hammer make a definite pendulum movement; strokes of equal force being used during the entire series of measurements on a beam.

For investigations at the Danish State Testing Laboratory requiring numerous tests a procedure allowing a rapid completion of the individual tests by a single person was required.

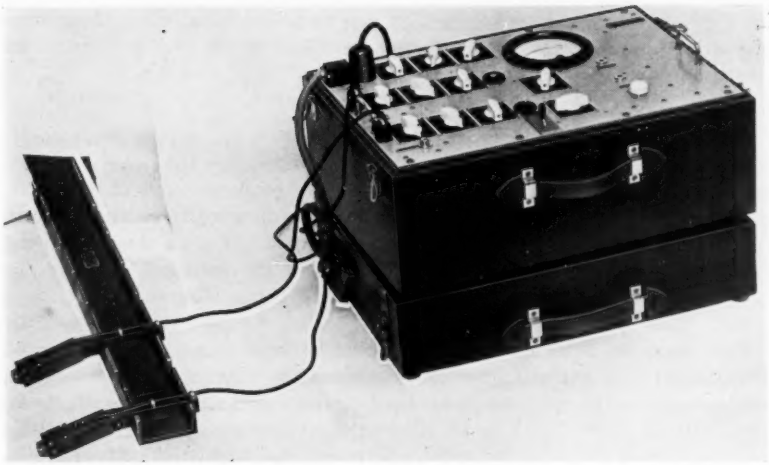

Fig. 4—Joergensen & Weibel condenser chronograph for measuring wave velocity in concrete

Guide beam for pick-ups

For routine testing of a large number of specimens it is important to have a simple and reliable arrangement to keep the two pick-ups in correct positions and a definite spacing. Test specimens at the Danish State Testing Laboratory were initially marked every 10 cm but this marking requires much time and locations were not exact, so a guide beam for the pick-ups was introduced.

The guide beam is a channel section, 110 cm long, provided with slots very exactly located every 10 cm. The pick-ups are mounted on brass rods fitting exactly into the slots. Vertical adjustment of the pick-ups is easy as the pick-up arms are hinged. The guide beam may be seen at the left in Fig. 4. Use of this guide beam makes marking of observation points on the specimen unnecessary. However, it should be certain that pick-up P_1 is at a constant distance from the point of impact when it is fixed at the outermost cut.

Electric hammer

For the preliminary investigations the pulse was produced by hand with a hammer. Later an electrically operated hammer simplified the procedure. An electromagnet attracts an iron core, and compresses a spring at the same time, the current being supplied by a 6-volt battery and is governed by a foot control. When the control is released, the magnet in turn releases the core which is pushed against the end of the specimen by the spring.

In this way light uniform blows are obtained and the spread of the data obtained is smaller than when blows are of varying force because the velocity measurement depends to a certain extent on the steepness and shape of the wave front and, consequently, on the energy of the impact. Experiments[6]

have shown that the force of the blow has to be increased when the volume of concrete specimens is increased; thus the blows of the electric hammer are too small for measurements on road slabs.

Measuring procedure

In the authors' investigations no influence from the supporting medium on the longitudinal wave velocity was observed. For the sake of safety, however, the test specimens were placed on felt strips throughout.

It proved preferable to place pick-up P_I at a distance twice the sectional dimension of the specimen from the place of impact. P_{II}, on the other hand, may be placed quite close to the other end of the specimen without any disturbance to the longitudinal wave velocity being observed by reflected waves.

Test beams 10 x 10 x 120 cm were used in most cases. P_I was placed 20 cm from the point of impact and remained there during the entire observation series on the specimen, whereas P_{II} was placed at distances of 10, 20, 30 ... 90 cm from P_I. Five readings of the tube-voltmeter were taken for each position of P_{II}, and the arithmetic mean was calculated. The f values measured are transformed into traveling times in micro-seconds through tables worked out on the basis of Eq. 4.

This arrangement and an experienced operator makes it possible to test a 120-cm beam in 3–5 minutes although as many as 45 readings may be required.

Appendix A describes in detail the further treatment of the test results, from which the average wave velocity c_l and the zero time t_0 may be found graphically or by the method of least squares. It is shown that the scatter of observations for a given pick-up spacing is less than 1 μ sec. A detailed statistical treatment of the test results provides an estimate of the inhomogeneity of the concrete.

From the wave velocity it is possible to compute the dynamic modulus of elasticity by well-known formulas, which, however, assume knowledge of Poisson's ratio and density of concrete. In addition the type of formula applicable to this method depends upon the sectional shape of the specimen. Poisson's ratio is difficult to determine and changes with time during curing as demonstrated by Jones.[3] In the present paper the wave velocity is used directly, as the computation of the modulus of elasticity, in the authors' opinion, has no advantages.

MANUFACTURE OF CONCRETE SPECIMENS

Materials

In the laboratory investigations ordinary portland cement (Type I) and Rapid cement were used, the latter being a high-early strength portland cement (Type III). For each of the cement types, mixes with 150 and 300 kg of cement per cu m of concrete were used.

The aggregates were of the type generally used in Copenhagen, *i.e.*, rounded coarse aggregate taken from the sea shores and consisting of varying propor-

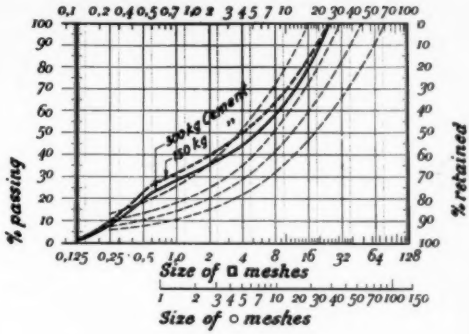

Fig. 5—Grading curves for concrete aggregates for test concretes containing 150 and 300 kg per cu m of cement

tions of quartzite, granite, flint and feldspar, while the fine aggregate is taken from the bottom of the sea and is composed mainly of quartz. The grading used is shown in Fig. 5.

The amount of water added was so regulated that one half of the series of test specimens had a slump of 5 cm, and the other half 15 cm.

Manufacture

Mixing was done in a 50-liter positive, rotating pan Eirich mixer. The materials were dry-mixed for about two minutes before adding the water, and thereafter was continued for three minutes. The mix was placed into four prismatic molds, 10 x 10 x 120 cm, and vibrated for one minute. A vibrating table with a frequency of 50 rps was used. The test specimens remained in the molds, covered with wet sacks until the following day. Then they were stripped and stored under water until tested.

RELATION BETWEEN WAVE VELOCITY AND CURING CONDITIONS

The water contained in the concrete has considerable influence on the modulus of elasticity found by the resonant frequency. This has been shown by Obert and Duvall,[7] Long and Kurtz[8] and Axon, Willis and Reagel.[9] Preliminary investigations with the condenser chronograph proved the correctness of the assumption of an interrelation of wave velocity in concrete and the percentage of water contained.

Fig. 6 shows the variations of wave velocity with age found for four beams of a series with 300 kg of portland cement per cu m of concrete, and $c/w = 1.67$. The specimens were stripped after 24 hours moist curing. Two of the beams were cured under water, and the other two at a relative humidity of 80 percent. The temperature was in both cases 18 C. It is seen that the wave velocity of the conditioned beams increases at a lower rate than for the water-cured beams, and that the wave velocity reached a constant level or decreased slightly after two weeks curing. This means that the decrease owing to drying exceeds the increase due to hardening. Beams were, therefore, cured under water until tested.

VARIATION OF LONGITUDINAL WAVE VELOCITY WITH AGE

A series of beams made for the preliminary investigations were cured under water at normal indoor temperature. Wave velocity tests were made frequently, and series of corresponding values of wave velocity and age were obtained.

The curves in Fig. 6 representing the relation between age and wave velocity were drawn estimatively. This procedure is satisfactory when many observations are at hand, and when the scatter of values is small.

An objective judgment on shape and location is obtained by a regression analysis by the method of least squares. A regression analysis requires a hypothesis with respect to the type of mathematical function that represents the interrelation of the two variables, *i.e.*, a hypothesis on the shape of the curve in general. Even in the case of a rather complex type of function it may, in many cases, be possible through appropriate transformation of the variables to apply an ordinary linear regression analysis.

From theoretical considerations supplemented by a series of trial calculations it proved convenient to apply a function of the type

$$c_l = \alpha + \gamma \times e^{-\beta/t} \quad \ldots\ldots (5)$$

where

c_l = longitudinal wave velocity in km per sec.
t = curing time in days
α, γ, and β are constants.

c_l is seen to be minimum, equal to α, when $t \to 0$. When $t \to \infty$ the curve has a horizontal asymptote

$$c_l = \alpha + \gamma$$

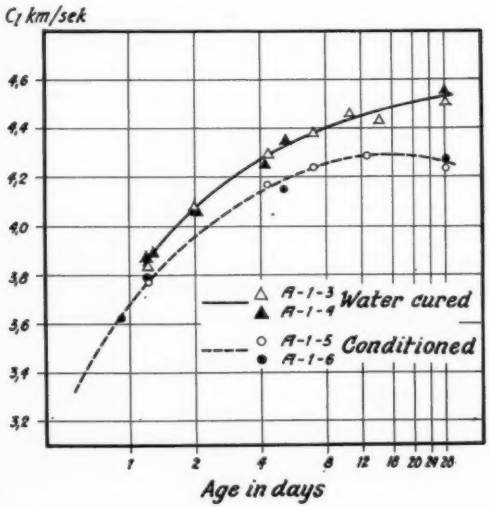

Fig. 6—Variation of longitudinal wave velocity with age. Specimens cured in water and in air

If the constant α has been found or estimated by other means computations are simplified considerably, the function being transformed into:

$$\log (c_t - \alpha^x) = \log \gamma - \beta/t \log_e (\alpha^x = \alpha) \quad \ldots\ldots (6)$$

i.e., the logarithm of the difference of the wave velocity in green concrete ($t = 0$), and at the moment of testing is a linear function of the reciprocal of the curing time. Hence, in plotting test results, where log $1/t$ values are abscissae and log $(c_t - \alpha^x)$ are ordinates, the hypothesis will involve an approximation of the points to a straight line. Based on a test series, estimative values of the two parameters, α and β, of the straight line could be so calculated that the sum of the squares of vertical distances from the marks to the line will be minimum.[10]

Attempts to measure the wave velocity of green concrete were unsuccessful, due partly to considerable damping of the impulse, and partly to difficulty in transferring the impulse from concrete to pick-ups. Green plastic concrete may be considered as a suspension of cement, sand, and gravel in water, in which the individual grains are enveloped by a film of water that prevents direct contact between the grains. Besides, a small quantity of air (1 or 2 percent by volume) is contained in the concrete.

Previously it was assumed that the velocity of propagation waves in the fresh concrete would have a lower limit determined by the wave velocity in water, *i.e.*, approximately 1.4 km per sec. Experiments in Paris[4] have shown that the velocity of wave propagation in cement paste immediately after mixing has a value of approximately 0.2 km per sec. This velocity slowly increases until initial set takes place and during the setting period the velocity increases rapidly. During hardening the wave velocity increases at a diminishing rate. The phenomenon of the low velocity in the initial period has been explained by L'Hermite. The theory is that one is not measuring velocity of the wave propagation, but the velocity of propagation of energy, where the solid particles suspended in the liquid have to transfer vibrating energy from one particle to another in a way analogous to coupled pendulums. The velocity of transmission of this vibrating energy between the particles is much smaller than the wave propagation in water.

The formulas for the relation between velocity and curing time are intended to cover only the hardening period, and by trial it was found that an initial velocity α of 1.5 km per sec was most suitable. According to the formula and initial value of the wave velocity of hardening concrete, the computed values of log $(c_t - 1.5)$ and $1/t$ are plotted in Fig. 7 for three different series of specimens with portland cement.

The points in Fig. 7 randomly approximate straight lines, so that the hypothesis applies to ages from 1 to 28 days. Extrapolation beyond this interval is not justified, however, especially as far as short periods are concerned, because the setting period may cause deviations from the hypothesis.

The composition of the three mixes is shown in Table 1, where the composition of four test series with Rapid cement is also given.

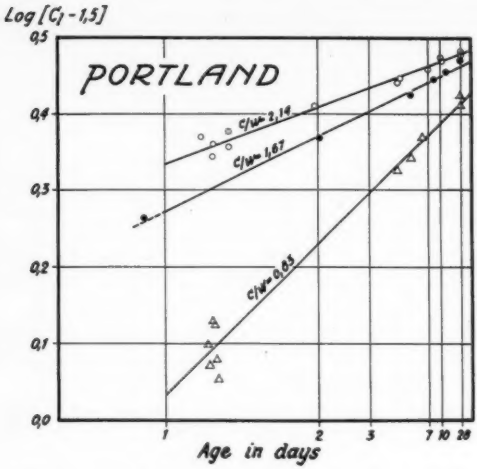

Fig. 7—Increase of longitudinal wave velocity of water cured specimens containing Danish portland cement. Inverted value of curing time as abscissa

The type of equation chosen involves a considerable scatter of the points in the graphic representation for short curing times. This applies, for example, to the points in Fig. 7 for c/w = 0.83. Observed velocities are given in Table 2 and the scatter of the results from the six beams is computed, the difference of the curing times being neglected. The coefficient of variation, v, of results from six beams from one batch amounts to 3 percent for tests after one day curing. This scatter is rather small compared with results of destructive tests on concrete after short curing periods.

TABLE 1—COMPOSITION OF TEST BEAMS AND EQUATIONS FOR WAVE VELOCITIES

Type of cement	Cement, kg per cu m	Water, l per cu m	c/w	w/c	log γ	β log$_e$	Eq. for c_t	c_t for $t = \infty$
Portland	300	140	2.14	0.47	0.485	−0.152	$1.5 + 3.06e^{\frac{-0.35}{t}}$	4.56
	300	180	1.67	0.60	0.470	−0.198	$1.5 + 2.95e^{\frac{-0.46}{t}}$	4.45
	150	180	0.83	1.20	0.429	−0.397	$1.5 + 2.69e^{\frac{-0.91}{t}}$	4.19
Rapid	300	170	1.77	0.56	0.457	−0.165	$1.5 + 2.86e^{\frac{-0.38}{t}}$	4.36
	300	190	1.58	0.63	0.455	−0.210	$1.5 + 2.85e^{\frac{-0.48}{t}}$	4.35
	150	180	0.83	1.20	0.436	−0.409	$1.5 + 2.72e^{\frac{-0.94}{t}}$	4.22
	135	195	0.69	1.45	0.434	−0.750	$1.5 + 2.71e^{\frac{-1.73}{t}}$	4.21

TABLE 2—OBSERVED VELOCITIES IN CONCRETE BEAMS

Age, hours	c_l, km per sec.	$\overline{c_l}$, km per sec.	$\Delta \times 10^2$	$\Delta^2 \times 10^4$
28	2.76		+ 1	1
28¼	2.68		− 7	49
28½	2.85	2.75	+ 10	100
28¾	2.84		+ 9	81
29	2.73		− 2	4
29¼	2.64		− 11	121

$$s = \sqrt{\frac{356}{5}} \times 10^1 = 0.084 \text{ km per sec}$$

$$v = \frac{0.084}{2.75} = 3 \text{ percent} \qquad 10^4 \times \Sigma\Delta^2 = 356$$

It will be apparent from Fig. 7 and Table 1 that an increase of w/c causes a reduced rate of development of the wave velocity, and the ultimate value is lower.

Fig. 8 represents the wave velocity in a coordinate system, where c_l is ordinates in ordinary arithmetic scale, and curing time is abscissae in logarithmic scale. Horizontal asymptotes, corresponding to wave velocity after infinitely long water curing, are shown in dot and dash lines, assuming the relation to apply beyond 28 days. The wave velocity for ages below 24 hours is indicated by dotted lines, the relation being assumed to apply also within this interval.

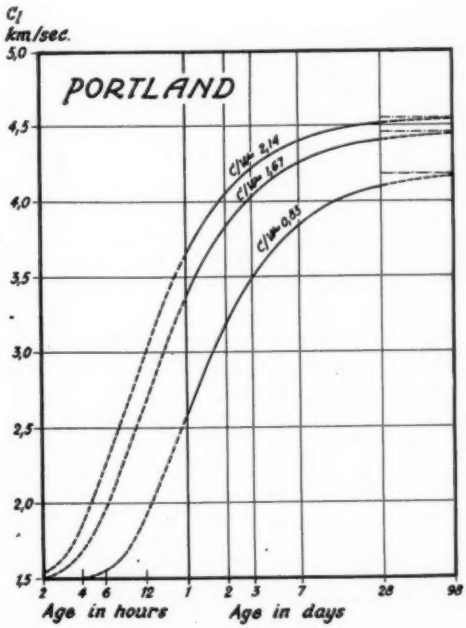

Fig. 8—Increase of longitudinal wave velocity of water cured specimens containing Danish portland cement. Curing time as abscissa in logarithmic scale

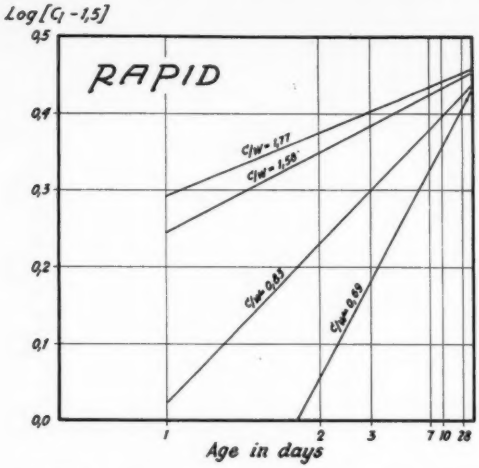

Fig. 9—Increase of longitudinal wave velocity of water cured specimens containing Danish high-early strength portland cement. Inverted value of age as abscissa

Table 1 contains values for 3 series with Rapid cement. The β and γ values are determined in the same way as for portland cement, *i.e.*, by the method of least squares. The curves are shown in Fig. 9. It will be seen from Table 1 that changes of w/c affect β considerably, whereas γ is more sensitive to changes of the aggregate-paste ratio. This applies especially to the Rapid series, where $\gamma + 1.5$, in column 8, corresponding to the wave velocity after infinitely long curing time, has almost equal values when the cement contents are equal. Halving of the cement contents, on the other hand, involves a rather considerable change of c_t for $t = \infty$.

Corresponding β and w/c values are given in Fig. 10. The points for portland cement approximate a straight line through the origin and $\beta = 0.75$ w/c, whereas the values for Rapid cement are scattered somewhat more. The deviation is particularly pronounced for β corresponding to w/c = 1.45.

The limited number of tests does not allow a definite statement on the applicability of the relation. Until further test results are at hand the follow-

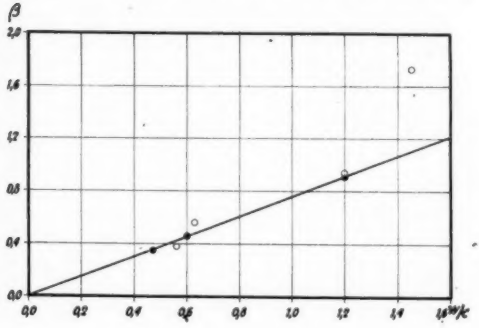

Fig. 10—Interrelation of the factor β and water-cement ratio. ●—portland cement. ○ – high-early strength cement

ing preliminary hypothesis may be used: $\beta = k \times w/c$, where k is a constant depending on the cement type and storage conditions for the cement. Besides, k will depend on the curing temperature of the concrete. Hence, β is intimately related to the progress of the process of hardening, while γ is thought to depend on the composition of the test specimen after the hardening has come to an end. γ is therefore expected to be dependent on the voids volume in the hardened concrete, the cement-aggregate ratio, and the elastic properties of the aggregates.

USE OF CONDENSER CHRONOGRAPH TO TEST OF CONCRETE IN SITU

The condenser chronograph is supplied with electric energy from dry cells and a storage battery and is consequently independent of local electrical sources. The apparatus is built into two strong cases and is well suited for use in the field. In the following the advantages of the condenser chronograph for field measurements are dealt with in general, together with minor changes in testing technique that may be desirable. Finally, investigations of various concrete structures are described.

Testing technique

For testing materials in place determination of the longitudinal wave velocity has the advantage that no definite shape of specimens is required, as in the case of measurement of resonant frequency. Observations have been made successfully on reinforced concrete structures such as decks, roads, runways, slabs cut from structures, and beams.

The individual observations require very little time so that the desired number of observations, generally five spaced at 10 cm, present no difficulty. The observation points are located on a straight line in the direction of the impact. Pick-up I is fixed not too close, usually 20 or 30 cm, from the point of impact, while pick-up *II*, by means of the guide beam (Fig. 1), is moved along in 10-cm steps as far as the vibration possesses sufficient energy to actuate the thyratron circuit. In some cases pick-up *II* has been fixed at the greatest possible distance from the place of impact and pick-up I moved stepwise towards the place of impact but obviously this did not cause any difference in the results. The concrete surface to be tested must be carefully cleaned as loose grains and dust may influence the results considerably.

When the concrete has an accessible and comparatively plane surface at right angles to the surface used for the observations, the impacts producing the vibrations may be applied to this surface. In the case of decks monolithically connected to other structural members, and without accessible surfaces suitable for application of the impulse, it is not necessary to produce such an accessible surface directly. At the center of a panel a hole may be drilled in which a steel rod is fixed by cement-sand mortar. Impulses are then applied through this rod. When time is limited, a sulfur-kaolin mixture may be used to fix the rod so that observations can be made immediately.

For observations on beams, where no essential damping of the vibrational energy occurs, a small electromagnetic hammer can be used to apply the

impulses. Comparatively weak impulses cause no error of importance. In concrete slabs, where the vibrational energy is spread into greatly increasing sections with distance from the place of impact, more powerful impulses are necessary, and a pendulum hammer is recommended.

Measurements of fire-damaged concrete

At a factory working with gasoline a two-story reinforced concrete building was constructed in 1913. It was divided into three sections: *A*, *B*, and *C*. An explosion took place on the ground floor of section *A*, and the concrete slabs of the deck above were damaged by the explosion, but the exposure to fire was of very short duration. In section *B* a sack storage caught fire. The concrete above this storage room was partly exposed to fire for several hours. In section *C* no essential damage was caused either by explosion or fire. In section *B* cracks and spalling of concrete in columns, beams and deck slabs were noticed.

It was desired to ascertain the extent of the damage by some testing method. After due consideration it was decided to determine the longitudinal wave velocity in intact and damaged concrete to obtain a comparative basis for judging the extent of damage.

The concrete slab had a thickness of 12 cm, and was supported by main and secondary girders spaced at 3.4 and 3 m, respectively, *i.e.*, the panels were 3.4 x 3 m. The slab had a two-way reinforcement of 8 mm diameter bars spaced at 8 cm and the cover was 1 to 2 cm.

At the center of panels to be investigated steel rods were fixed to transmit the impulses. Testing points were marked by lines in the longitudinal direction of the building, and when possible also in the transverse direction. The asphalt flooring was removed on both sides of the lines, and the concrete surface was carefully cleaned. For application of impulses a pendulum hammer was used. Observations were made at 9 points of each line with 5 observations at each point.

In section *A* an average wave velocity of 3.45 km per sec was found with a standard deviation of 0.28 km per sec as a result of 6 observations. In section *C* average wave velocity determined from 4 individual observations was 2.85 km per sec with a standard deviation of 0.09 km per sec. In section *B* the area was divided into 2 parts, *viz.*, one area including approximately 50 percent of the total deck area, where the results indicated that the concrete was so heavily damaged that it had to be replaced. As a result of 40 observations an average velocity of 2.16 km per sec was obtained. The standard deviation was in this case 0.38 km per sec. The adjoining area of section *B* had an average wave velocity of 2.98 km per sec and a standard deviation of 0.29 km per sec. In this case 16 observations were recorded, each of which was determined from a total of 45 readings of f values as previously described (Table 3).

The wave velocity was considerably lower in section B_1, the section where the concrete later was replaced. The wave velocity in section B_2 was about

TABLE 3—SUMMARY OF RESULTS FROM A FIRE-DAMAGED CONCRETE STRUCTURE

Section	Numbers of observations	Average wave velocity, $\bar{c}_l$, km per sec	Standard deviation s_c, km per sec	Remarks
A	6	3.45	0.28	Place of explosion
B_1	40	2.16	0.38	Exposed to fire for several hours
B_2	14	2.98	0.29	Exposed to fire for a shorter period
C	4	2.85	0.09	Not damaged

the same as in section *C*, but the variation is considerably higher, indicating a much lower homogeneity. Wave velocity even for the undamaged concrete in this old building was considerably lower than for similar concrete cured for 28 days, where the wave velocity is 3.7–3.9 km per sec. depending on the cement content and initial water content.

Frost damaged concrete floor

A concrete floor slab was exposed to freezing without adequate protection immediately after placing. Consequently, the coarse aggregate at the surface was loose, and the mortar had obtained very little strength. When hit by a hammer the concrete had a dead ring. From inspection the impression was, however, that the damage was only superficial, and the concrete was consequently subjected to an investigation by the chronograph after approximately one month curing at low temperature. The measurements were made on the top and bottom surfaces of the slab. In the latter case the pick-ups were fixed to the guide beam and kept in contact with the concrete by small spiral springs. Results are given in Table 4.

It will be noted that the wave velocity is considerably higher at the bottom than at the upper face and approximately that expected of this type of concrete type after the normal curing period.

The factor of inhomogeneity, s_a, in Table 4 is computed according to formulas stated in Appendix A. It is primarily an estimate of the variation in wave velocity within the measuring length. For normally-cured undamaged concrete manufactured in the laboratory the factor of inhomogeneity is approximately 1 μ sec. In this case the factor of inhomogeneity is rather

TABLE 4—WAVE VELOCITY OF FROST DAMAGED FLOOR SLAB

		Wave velocity, c_l, km/sec	Factor of inhomogeneity, s_a, μ sec	Uncertainty of observations $\bar{s}_l$, μ sec
Upper face	1	3.14	14.3	0.3
	2	3.25	16.9	0.8
	Mean	3.20	16.0	0.6
Bottom face		3.79	6.6	1.2

TABLE 5—INVESTIGATION OF A CONCRETE GIRDER

Mean wave velocity, c_l	3.17 km per sec
Uncertainty of observations, s_t	0.2 μ sec
Factor of inhomogeneity, s_a	8.6 μ sec
Mean density of 20 x 20 x 20-cm cubes	2.14 $\pm$ 0.01 kg per l
Mean compressive strength	108 $\pm$ 5 kg per sq cm

high in the bottom face, and the upper face of the slab has values more than twice this value. This is probably due to lack of bond between mortar and coarse aggregate.

The uncertainty of observations, $\bar{s}_t$, is computed according to formulas developed in Appendix A. $\bar{s}_t$ makes it possible, by comparison with s_a, to determine to what degree the variations in wave velocity are due to the apparatus.

Measurements on a concrete girder

At a building enterprise with only occasional supervision the quality of a concrete girder was supposed to be lower than specified, and consequently it was decided to investigate the concrete quality by the wave velocity method and by compressive tests on 20 × 20 × 20-cm cubes sawed out of the concrete member. The results are given in Table 5.

All these determinations are in accordance with each other and indicate that the quality of the concrete was considerably lower than the specifications, which called for a concrete with a compressive strength of 250 kg per sq cm. A simple relation between compressive strength and wave velocity does not exist, but experience gained in the laboratory from routine testing of a large number of control beams delivered from building sites in the vicinity of Copenhagen indicates that concrete of this type with a rather high water content, say 200 l per cu m should have a wave velocity of 3.8 km per sec and a s_a value of 2 μ sec after 28 days curing in air.

Concrete pavements

Danish specifications for concrete pavements call for control of the concrete quality during construction by determination of the compressive strength of 20 x 20 x 20-cm cubes and the modulus of rupture of 10 x 15 x 80-cm beams. Furthermore the pavement is investigated after completion by drilling of 6-in. diameter cores, whereby it is possible to control thickness of the pavement and obtain compressive strength of the cylinders and voids volume of approximately 500 g specimens taken from top and bottom of the broken cylinder.

The contract usually contains elaborate formulas for computing penalties and reductions in the contract sum or extension of the guarantee period from five to seven years. These measures are taken when the thickness or the cylinder strength is deficient. The drilling technique has, however, in a number of cases been inadequate so that the compressive strength results have shown a great scatter with several test results below the minimum

TABLE 6—TEST DATA FROM RUNWAY TEST SLABS

Test Slab	Wave velocity, km per sec		Number of observations	Compressive strength, kg per sq cm		Number of observations
	$\bar{c}_l$	s_c		$\bar{\sigma}$	s_σ	
A	4.59	0.12	28	450	14	3
B	4.35	0.17	30	450	21	3

strength. This has been especially true when the coarse aggregate has had a maximum size of 3 in. and when the pavement thickness has been greater than 8 in.

There has consequently been a need for new testing methods for determining concrete quality of completed pavements. Following are some results obtained in the field on concrete pavements. Due to security reasons a great number of test results from airports have been classified and cannot be published at present.

Measurements on test slabs—After completion of runway extensions at a civil airport, a number of the cylinders showed too low cylinder strengths. To demonstrate that the concrete quality nevertheless was sufficient, the contractor cut out 1 x 1-m test slabs 30 cm thick in the vicinity of the cylinder holes, where the compressive strength of the cylinders had been too low. A grid of measuring lines 10 cm in each direction was drawn on the test slab, and the wave velocities were determined within the area, where later on three 30-cm cubes were cut out and tested in compression. Results are stated in Table 6.

Mean velocity was computed as an average of 28 and 30 single observations on a measured length of 10 cm, respectively, as only part of the grid covered the concrete from which the cubes were cut. The standard deviations of the wave velocities stated consequently apply to much shorter lengths than normally employed and are accordingly higher. An estimate of s_a for test slab A is 0.6 μ sec and for test slab B is 0.9 μ sec.

The wave velocities are those of rather high quality concrete, as concrete with a cement content of 300 kg per cu m and a water-cement ratio below 0.5 normally gives wave velocities above 4.20 km per sec after 28 days curing. The indicated compressive strength is considerably higher than anticipated from the test cylinders, which both had shown computed compressive strengths of 15 x 30-cm specimens lower than 250 kg per sq cm after 2 months curing. It should also be noted that the standard deviations of wave velocity and compressive strength are higher for test slab *B* than for test slab *A*.

Concrete Runway Tested in Situ—Many measurements have been made on concrete pavements in the last three years and the results from a concrete taxiway are given as an example. The pavement was constructed during the German occupation and no data with regard to mix proportions and other characteristics are available, but it is known that this type of work

TABLE 7—WAVE VELOCITIES OF A CONCRETE PAVEMENT

Slab No. 1			Slab No. 2			Slab No. 3		
Station	c_l, km per sec	s_a, μ sec	Station	c_l, km per sec	s_a, μ sec	Station	c_l, km per sec	s_a, μ sec
2006	4.25	2.6	2006	4.27	3.1			
			2040	4.57	3.4	2040	4.27	3.6
2080	4.20	5.6	2080	4.32	2.1			
			2120	4.62	2.5	2120	4.13	2.7
2160	4.39	1.8	2160	4.35	5.0			
			2200	4.26	6.0	2200	4.22	5.4
2240	4.21	3.1	2240	4.19	3.6			
			2280	4.38	4.6	2280	4.20	2.7
2320	4.24	0.8	2320	4.11	6.2			
			2360	4.21	2.8	2360	4.23	1.8
2400	4.32	1.9	2400	4.26	4.0			
			2440	4.26	2.2	2440	4.30	1.4
2480	4.06	5.9	2480	4.33	1.6			
$\bar{c}_l$ = 4.24 km per sec s_c = 0.10 km per sec $\bar{s}_a$ = 3.6 μ sec			$\bar{c}_l$ = 4.32 km per sec s_c = 0.14 km per sec $\bar{s}_a$ = 3.9 μ sec			$\bar{c}_l$ = 4.23 km per sec s_c = 0.06 km per sec $\bar{s}_a$ = 3.2 μ sec		

usually is inferior to pavements constructed in peace time. Results appear in Table 7.

In slabs 1 and 3 determinations were made at approximately 80-m intervals, and in slab 2 at 40-m intervals. For each measuring length 10 different pick-up spacings were used, and for each spacing 5 readings were taken.

There is little difference in the average wave velocity for the three slab rows. The results show, however, that slab 3 has the lowest between-profile standard deviation of the wave velocity and the lowest value of $\bar{s}_a$, the latter being an estimate of the within-profile variation.

CONCLUSIONS

1. The condenser chronograph measures time intervals for a fixed pick-up spacing with a standard deviation below 1.0 μ sec.
2. From the observations it is possible to determine the average wave velocity either graphically or by computation.
3. If the average wave velocity is presented as a straight line, the individual points representing the average traveling times for different pick-up spacings will be distributed at random about the straight line.
4. Over-all estimates of the deviations from the straight line are in most cases larger than justified by the scatter of the readings.
5. The over-all estimate of variation in wave velocity is presumed to reflect inhomogeneity of the concrete.
6. Variation in wave velocity within a profile is much greater for concrete previously exposed to damaging influences such as frost and fire than for normally cured concrete.
7. The wave velocity method is suitable to follow changes in concrete due to hardening, and a hypothesis of the relation between increase in wave velocity after final set of water-cured concrete beams and curing-time is represented.
8. It is shown that water cured specimens obtain high wave velocity more rapidly than specimens cured in air of 80 percent relative humidity.
9. It is expected that different treatments of the concrete may be reflected in the wave velocity.

10. The method used has proved to be suitable for measuring wave velocity in actual structures, whereby a discrimination between concrete of high and low quality may be made.

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CALCULATION OF WAVE VELOCITY

As previously described, 5 determinations of the residual voltage of the condenser are made for each pick-up spacing, and in investigating a 1.20-m beam 45 readings are, thus, to be made, the distance from the first pick-up to the point of application of the pulse always being 20 cm. For each pick-up spacing the arithmetical average is computed, and the readings are transformed into micro-seconds by means of a table worked out with due consideration of the characteristics of the particular apparatus used. The subsequent treatment of the results may proceed in various ways.

Graphic method

By the graphic method corresponding values of pick-up spacing and transmission time are plotted in a coordinate system, spacings as abscissae and times as ordinates. The points thus obtained approximate a straight line. This line is so drawn that all the vertical distances from the marks to the line are, estimatively, as small as possible (Fig. 11). The inclination of the line represents the average wave velocity. The line intersects the ordinate axis at a point with ordinate t_0 because the two circuits are not equal, electrically. This zero-time corresponds to zero pick-up spacing. The zero-time depends on the amplification in the circuits and can, therefore, not be determined once for all. This fact does not mean anything to the applicability of the method, however.

The graphic method possesses the advantage of affording a possibility to localize cracks in the concrete indicated by parallel displacement of this line or other irregularities. The

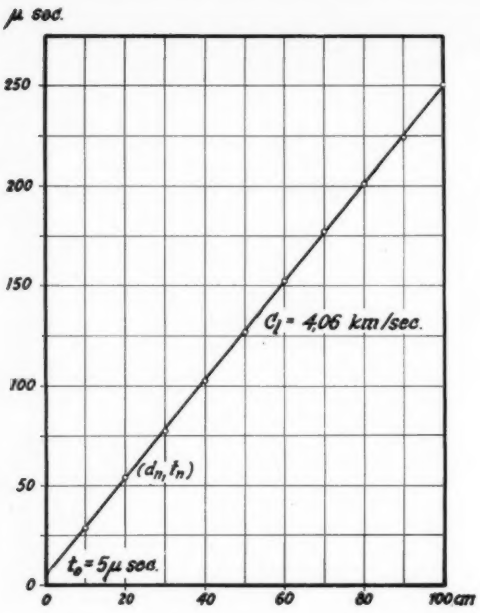

Fig. 11—Relation between traveling times and pick-up spacings

method requires considerable exactitude in plotting the points and, involves of course, a certain arbitrariness in drawing the straight line.

Method of least squares

By this method the aforementioned straight line is so determined that the sum of the vertical distances of the marks from the line is a minimum.

Each of the m sets of pick-up spacing and wave velocity corresponds in this coordinate system to a point (d_n, t_n), whereby $n = 1, 2, 3 \ldots\ldots m$, and t_n is the time recorded, in micro-seconds, corresponding to a pick-up spacing d_n, expressed in cm.

The straight line is expressed by

$$t = k + \frac{1}{c_l}(d - \bar{d}) \quad \ldots\ldots (7)$$

Where

t = transmission time corresponding to an arbitrary pick-up spacing, d

$\bar{d} = \frac{1}{m}\sum_{n=1}^{m} d_n$, *i.e.*, the average of the pick-up spacings

c_l = the unknown wave velocity, in cm per micro-second

k = an unknown constant

The condition

$$F\left(k \frac{1}{c_l}\right) = \sum_{n=1}^{m} [t_n - (k + \frac{1}{c_l}(d_n - \bar{d}))]^2 = \text{minimum} \quad \ldots\ldots (8)$$

affords two equations for determination of c_l and k, when

TABLE 8—EXAMPLE OF COMPUTATION

Distance, cm	$f \times 10^3$ Observation No. 1	2	3	4	5	$\bar{f} \times 10^4$	$s_f \times 10^4$	$20 \times t_n$, μ sec	$20 \times s_t$, μ sec	$20^2 \times s_t^2$, μ sec^2	$20 \times t_n'$, μ sec	$20 \times \Delta$, μ sec	$20^2 \times \Delta^2$, μ sec^2
1	2	3	4	5	6	7	8	9	10	11	12	13	14
10	893	889	886	884	884	8872	38	599	23	529	619	− 20	400
20	790	794	792	798	811	7970	84	1135	50	2500	1109	+ 26	676
30	732	732	730	731	732	7314	9	1564	6	36	1600	− 36	1296
40	650	651	654	656	653	6528	24	2133	19	361	2090	+ 43	1849
50	596	592	591	592	598	5938	30	2607	24	576	2581	+ 18	324
60	537	536	540	546	541	5400	39	3081	35	1225	3072	+ 9	81
70	494	495	499	494	492	4948	26	3518	26	676	3562	− 44	1936
80	444	447	456	448	446	4482	46	4013	51	2601	4053	− 40	1600
90	403	402	402	404	402	4026	9	4550	11	121	4543	+ 7	49
100	362	364	364	364	363	3634	9	5061	13	169	5034	+ 27	729

$\bar{d} = 55$ $\quad \Sigma dt = 97{,}952$ $\quad \Sigma t = 1413.1$ μ sec $\quad \Sigma s_t^2 = \frac{8794}{20^2}$ μ sec^2 $\quad \Sigma\Delta^2 = \frac{8940}{20^2}$ μ sec^2

$\Sigma d^2 = 38{,}500$ $\quad \bar{t} = 141.31$ μ sec

$$\frac{\delta F}{\delta k} = 2k \times m - 2 \sum_{n=1}^{m} t_n = 0 \quad \ldots\ldots (9)$$

$$\text{and } \frac{\delta F}{\delta \frac{1}{c_l}} = 2 \times \frac{1}{c_l} \sum_{n=1}^{m} (d_n - \bar{d})^2 - 2 \sum_{n=1}^{m} t_n(d_n - \bar{d}) = 0 \quad \ldots\ldots (10)$$

Hence

$$k = \frac{1}{m} \sum_{n=1}^{m} t_n = \bar{t} \quad \ldots\ldots (11)$$

$$\text{and} \qquad c_l = \frac{\sum_{n=1}^{m} d_n^2 - m \times \bar{d}^2}{\sum_{n=1}^{m} t_n d_n - m \times \bar{d} \times \bar{t}} \quad \ldots\ldots (12)$$

When the constants t and c_l are found the zero time t_0 of the apparatus is found in microseconds through insertion of $d = 0$ and $k = \bar{t}$ in Eq. 7.

$$t_0 = \bar{t} - \frac{1}{c_l} \bar{d} \quad \ldots\ldots (13)$$

As a check on the calculation, the longitudinal wave velocity is found from the transmission time for maximum and minimum spacing, d_m and d_1, of the pick-ups.

$$c_l = \frac{d_m - d_1}{t_m - t_1}, \quad \ldots\ldots (14)$$

where t_m and t_1 are the corresponding time readings.

Normally, there will be little difference between the wave velocity found by the last method and the velocity found by one of the other methods.

In Table 8 an example is given which shows to which extent agreement of the individual observations is obtainable, and how to make the calculations. The results are from an arbitrarily chosen test.

Substituting into Eq. 12 and 13:

$$c_l = \frac{38500 - (10 \times 55^2)}{97952 - (10 \times 55 \times 141.31)} = \frac{8250}{2022.6} = 0.4077 \text{ cm per } \mu \text{ sec} = 4.08 \text{ km per sec}$$

$$t_0 = 141.31 - \frac{55}{0.408} = 6.5 \ \mu \text{ sec}$$

Checking by Eq. 14:

$$c_l = \frac{90}{253.1 - 30.0} = \frac{90}{223.1} \text{ cm per micro-sec} = 0.403 \text{ cm per } \mu \text{ sec} = 4.03 \text{ km per sec}$$

By the graphic method* $c_l = 4.06$ km per sec.

Computation of standard deviations $\bar{s}_t$ and s_a

The standard deviations of the f values in column 8 of Table 8 were computed according to the formula

$$s_f = \sqrt{\frac{\Sigma (f - \bar{f})^2}{n - 1}} \quad \ldots\ldots (15)$$

These standard deviations have been transformed to the standard deviations, s_t of the traveling times in micro-seconds. The values of s_t are given in column 10 of Table 8. This procedure is justified as the deviations are so small that a short part of the curve of the relation between measured f values and the traveling times can be substituted by a straight line without appreciable errors.

From the common estimate of the zero-time and the average wave velocity the adjusted traveling times were computed assuming rectilinear relation between pick-up spacing and traveling time. The values are given in column 12, Table 8.

The differences Δ of observed traveling times in column 9 and the adjusted traveling time in column 12 are given in column 13. This difference will, as in Fig. 11, give the vertical distance between the straight line and the point corresponding to the actual observed traveling time computed from the mean of 5 observations. Now the question is, whether the variation of the 5 single values is of such an order that we cannot reject the hypothesis that the mean value might as well be situated on the straight line. The standard deviation of the mean is computed by dividing s_t by $\sqrt{n}$, where n is the number of observations, in this case $n = 5$. The small number of observations for each pick-up spacing allow only in few cases to reject with any significance that the mean may have a value coincident with the adjusted value.

If similar consideration is used for the total measuring length, the number of observations become, however, so large that the possibility increases of rejecting the hypothesis that the deviations from the straight line are only due to variations in the observed f values. The common estimate of the standard deviation of the mean values $\bar{s}_t$ is computed according to the formula

$$\bar{s}_t = \sqrt{\frac{\Sigma s_t^2}{m \times n}} \quad \ldots\ldots (16)$$

where n is the number of pick-up spacings and m the number of observations for a given pick-up spacing. The computation of $\bar{s}_t$ is independent of the shape of the regression curve, while the computation of the common estimate of the average value deviation from a straight line assumes that the regression curve is rectilinear. The computation of s_a is carried out according to the formula

$$s_a = \sqrt{\frac{\Sigma \Delta^2}{n - 2}} = \sqrt{\frac{\Sigma (t_n' - t_n)^2}{n - 2}} \quad \ldots\ldots (17)$$

If the condition

*Fig. 11 is based on different test results than those stated in Table 8.

$$\frac{s_a^2}{\bar{s}_t^2} > F_{.05\,(n-2,4n)} \quad \ldots\ldots (18)$$

is fulfilled there is a greater scatter of the points plotted from the average traveling times than implied by the scatter of observations. The quantity $F_{.05\,(n-2,4n)}$ is a 5 percent level of significance for the distribution of the stated variances, and $n - 2$ and $4n$ are numbers of degrees of freedom in nominator and denominator, respectively, for n pick-up spacings. Values of $F_{.05\,(n-2,4n)}$ may be found in statistical tables.

In the above-mentioned example of computation we find by inserting in Eq. 16

$$\bar{s}_t = \sqrt{\frac{8794}{20^2 \times 5 \times 10}} = \sqrt{0.4397} = 0.7\ \mu \text{ sec}$$

and by inserting in Eq. 17

$$s_a = \sqrt{\frac{8940}{20^2 \times 8}} = \sqrt{2.795} = 1.7\ \mu \text{ sec}$$

and finally by inserting in Eq. 18

$$\frac{s_a^2}{\bar{s}_t^2} = \frac{2.795}{0.4397} = 6.4 > F_{.05\,(8,40)} = 2.18$$

The computations show that a significant difference exists between the two estimates of scatter, and consequently the hypothesis that the traveling time is a rectilinear function of the pick-up spacing cannot be maintained, *i.e.*, the observed velocities vary along the measuring length. As stated[6] there may be several different causes for this variation, but it is supposed that the main cause is due to the inhomogeneity and anisotropy of the concrete, and consequently the quantity s_a is, in this paper, assumed to be an estimate of the inhomogeneity of the concrete.

The components of the concrete have different wave velocities. The wave velocity of the aggregates is independent of time and has in the authors' experiments been computed to be about 4.6 km per sec, while the wave velocity of the cement paste increases with time during water curing at normal temperature. The ultimate wave velocity of the cement paste depends upon the water-cement ratio, air content and other factors but will in most cases be lower than the wave velocity of the aggregates. As the diameter of the coarse aggregate is large in comparison with the distance between two consecutive pick-up positions the distribution of the coarse aggregate will cause deviations from the straight line which illustrates the average wave velocity.

As a measure of these deviations the authors have used the standard deviation, estimated by calculating s_a. The magnitude of the estimate s_a will decrease as the difference between wave velocity of aggregate and cement paste decreases, *i.e.*, with curing time. This explains why the factor of inhomogeneity s_a in a test series decreased from 15 μ sec to 1.0 μ sec from 1 days curing to 28 days curing.

Analogous theory explains the increase of s_a during deterioration of the concrete, whereby the bond between mortar and aggregate is destroyed and the cement paste is damaged.

Discussion of a paper by Johannes Andersen and Poul Nerenst:

Wave Velocity in Concrete*

By R. JONES, J. R. LESLIE and V. R. STURRUP, and AUTHORS

By R. JONES†

The condenser chronograph method is convenient for measuring wave velocity on the surface of solids and the authors showed how admirably it can be applied to studying surface deterioration or damage of concrete. The writer believes, however, that it is necessary to realize the limitations imposed by the systematic variations which may exist within concrete specimens.

For example, in a road slab compacted by surface vibration, the wave velocity is usually highest near the top and may fall to a very low value near the bottom if this is left relatively uncompacted. Even in thin slabs anomalous effects may occur, as we found recently when testing a 5-in. floor slab which had been vibrated against steel forms. The method we employ is to time a pulse of ultrasonic vibrations of 60 or 200 kc per sec frequency either along the surface or through the thickness of the slab[1,2] to an accuracy better than ± 1 percent. Wave velocities along the top surface, along the bottom surface and through the thickness were 14,700, 14,350, and 13,800 ft per sec, respectively. The discrepancy is attributed to the slab having a higher density near the top and bottom surfaces than in the middle. Such a form of density variation has been detected by a gamma ray technique[3] on slabs in which the compacting vibrations are reflected at the bottom form.

There are also cases in which deterioration of concrete is located within the mass and would not be apparent from surface measurements. One example arises from the effect of heat on concrete made with high-alumina cement when the lift thickness is excessive. Fig. A gives contours of transmission times in microseconds per ft (*i.e.*, the reciprocal of the wave velocity) within a 3-ft cube and shows how the deterioration is most marked near the center. These tests were made in collaboration with Lafarge Aluminous Cement Co., Ltd., which made the cubes, and it is because of this effect that the recommended lift thickness of high-alumina cement concrete is normally restricted to about 18 in.

The authors have shown how the scatter of observations is related to the inhomogeneity of concrete and how this is increased when the concrete is damaged, or when it has not hardened properly. This represents a valuable

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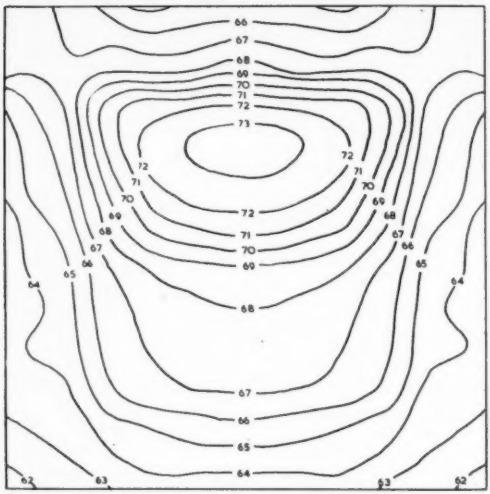

Fig. A—Contours of transmission times within 3-ft cube of high-alumina concrete (mean curves assuming symmetry about two vertical planes through axis and parallel to the faces)

observation and emphasizes the value of an adequate analysis of the results on which the authors are to be congratulated.

Following their lead we have analyzed some recent results on 4-in. test cubes, six of which were tested at ages of 1, 3, 7, 14, and 28 days by the ultrasonic pulse technique. Due to the method of preparation of the cubes, we find different distribution curves of the transmission times across the top, middle and bottom thirds of the cube, and the 18 observations on each cube were separated into these divisions. Table A gives the mean velocity, mean transmission time, and standard deviation of the transmission time in the top and bottom sections of the cubes at the different ages. The decrease in scatter of the results as the concrete hardens is in accordance with the results given by the authors. The difference in wave velocity in the top and bottom sections has already been discussed,[4] and this further analysis appears to indicate a difference in the hardening properties also consistent with the presence of a higher percentage of free water in the top section of the cube.

The problem of applying the wave velocity method to the testing of structures is largely one of knowing how to interpret the result. Apart from moisture content, wave velocity is dependent upon such factors as type and percentage of coarse aggregate, which have little or no effect on compressive strength. The presence of calcium chloride, on the other hand, affects the compressive strength to a relatively greater extent than the wave velocity. To overcome these difficulties we have usually made auxiliary measurements on test cubes of the same concrete mix as used in the structure.[5] While this procedure has disadvantages, it does serve to relate the wave velocity with a standard which, rightly or wrongly, is an accepted criterion of quality.

TABLE A—MEASUREMENTS ON 4-IN. TEST CUBES OF A 1:6/0.60 CONCRETE

Age, days	Top section of cubes*			Bottom section of cubes*		
	v, fps	t, micro-seconds	s_t, micro-seconds	v, fps	t, micro-seconds	s_t, micro-seconds
1	13,600	24.5	0.51	14,100	23.6	0.46
3	14,800	22.5	0.44	15,200	21.9	0.33
7	15,200	21.9	0.37	15,550	21.4	0.27
14	15,450	21.5	0.33	15,800	21.1	0.29
28	15,550	21.4	0.25	16,000	20.8	0.24

*Where v is the mean wave velocity; t is the mean transmission time; and S_t is the standard deviation of the transmission time.

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By J. R. LESLIE and V. R. STURRUP*

The authors are to be commended upon their contribution to the field of nondestructive testing of concrete and also their mathematical approach to the question of setting time and inhomogeneity of concrete.

Their statement that velocity measurements depend on the steepness of the wave front and the energy of the impact (p. 617) might suggest difficulties in duplicating results with various observers and instruments. Possibly the authors could give details of the percentage change induced in this regard.

The observations that the velocity in a fresh mix is less than that in water has also been observed in Soniscope tests. By casting the transducers into the concrete it was possible to measure the pulse velocity almost immediately after mixing. The velocity was found to remain nearly constant at a value of 0.61 km per sec (2000 fps) during the first 3½ hr after mixing.

Experience has shown that velocity measurements along a surface do not necessarily reflect the quality of the interior concrete. For this reason, velocity measurements through the structure are normally to be preferred to surface transmissions. The authors do not state explicitly how transmissions through a specimen may be made with their instrument.

The paper emphasizes the need for further studies of pulse propagation in concrete. The variation of attenuation with frequency, the presence of dispersion, and the relationship to Poisson's ratio would all provide a fruitful field of research. Such information may aid in the explanation of the processes of hardening and deterioration of concrete.

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AUTHORS' CLOSURE

With reference to the discussion by Messrs. Leslie and Sturrup, the authors should like to comment upon the influence of steepness of the wave front on the recorded traveling times. In Fig. B it is assumed for convenience that the wave front can be illustrated by a straight line. Curve "a" is produced by a more powerful impact than curve "c," and the steepness of curve "a" is greater. Due to passage through the material, the amplitude of the longitudinal displacement will decrease to a certain extent, due to damping, and the wave front will be less steep.

Actual traveling time is represented by the distance between points R and S, which indicate the first infinitesimal displacement. Impulses transmitted from the pick-ups cannot attain sufficient force until the displacement has reached a certain value x_p. The reaction of the pick-up, P_I, is delayed t_a thereby. On arrival of the impulse at the pick-up, P_{II}, the delay is $t_a + \Delta t_a$, and the traveling time recorded is increased by Δt_a, because of the change of the wave front produced by damping ($x_{a,2} < x_{a,1}$). Hence the velocity observed will be lower than the theoretical velocity.

When an extraordinarily low impact energy is applied, the maximum amplitude may be of the same magnitude as x_p, as indicated by curve "c." Therefore t_c will exceed t_a and, moreover, Δt_c will increase. It should, however, be borne in mind that the actual wave fronts are not straight lines but sinusoidal curves, which means a considerable increase of Δt_c, because the sensitivity level will intersect the flat part of the wave front. Hence low impact energy applied to heavy concrete masses will involve prolonged traveling times. It is furthermore obvious that small variations of the maximum amplitude in the vicinity of x_p will cause considerable variations of Δt_c and thereby increase the scatter of the readings and influence the standard deviation $\bar{s}_t$.

Finally we may have a specimen with great damping effect, caused by either the size and shape or by low quality of the concrete. This situation

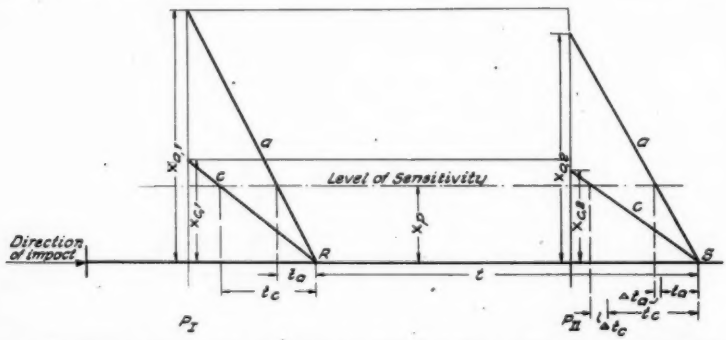

Fig. B—Steepness of wave fronts is changed by damping, causing an increase in traveling time the magnitude of which depends on the level of sensitivity and impact energy

may be illustrated by assuming that curve "a" at pick-up P_I is changed to curve "c" at pick-up P_{II}. In this case the recorded traveling time will be $t_c + \Delta t_c - t_a$ greater than the true traveling time, and thereby influencing the wave velocity to a comparatively high degree.

It is seen from Fig. B that Δt could be reduced through a reduction of x_p, *i.e.*, through improved sensitivity of both pick-ups.

In the condenser chronograph it is possible to vary the amplifications in the two circuits independently, and in ordinary investigation of beams the maximum amplitude—even with the use of the small spring hammer—is about 50 times x_p and damping cannot change the lower part of the wave front to a noticeable degree.

In a limited investigation on concrete pavements the effect of a pendulum hammer weighing about 300 g was compared to a small electric spring hammer used in the laboratory. It was found that the velocities determined by the small electric hammer were approximately 10 percent lower than when determined by the pendulum hammer. At the same time the standard deviation $\bar{s}_t$ increased from 0.5 microseconds to 2.1 microseconds, presumably due to the damping in the 7-in. concrete slab. With the small electric hammer the readings at the greatest pick-up spacing, 100 cm, suddenly corresponded to the velocity of slower waves, presumably Rayleigh waves, showing that the amplitude of the longitudinal waves was below the level of sensitivity.

These results show that the energy of the impact should be increased when the volume of concrete is increased so that the amplitude will not fall below a certain critical value, which depends upon the sensitivity of the pick-ups and the damping of the material investigated. As already pointed out, the level of sensitivity is 1/50 of the amplitude obtained by the use of the spring hammer on beams. Furthermore, the condenser chronograph is so constructed that it is possible to determine the point where the level of sensitivity is coincidental with the amplitude. When this has been determined, the sensitivity may be increased, for example, 50 times, thereby insuring that the measurements are taking place at the bottom of a steep wave front. In the laboratory it has not been possible to obtain any significant variation of wave velocity due to variation in the energy of the impact, but the scatter of the readings for a given pick-up spacing can be somewhat diminished by the use of uniform blows. This relates to concrete specimens which are undamaged and at ages above one day. In concrete specimens with greater damping, wave velocities may be influenced a few percent, and to use an energy of impact sufficient for all types of specimens a pendulum hammer weighing 100 g is now used instead of the electric spring hammer.

Messrs. Jones, Leslie and Sturrup state that velocity measurements along a surface do not necessarily reflect the quality of the interior concrete. By investigations on metals referred to in *Building Research Report* No. 3* it

*Andersen, Johannes, Nerenst, Poul, and Plum, Niels M., "The Non-Destructive Testing of Concrete with Special Reference to the Wave Velocity Method," *Building Research Report* No. 3, The Danish National Institute of Building Research, Copenhagen, 1950.

AUTHORS' CLOSURE

With reference to the discussion by Messrs. Leslie and Sturrup, the authors should like to comment upon the influence of steepness of the wave front on the recorded traveling times. In Fig. B it is assumed for convenience that the wave front can be illustrated by a straight line. Curve "a" is produced by a more powerful impact than curve "c," and the steepness of curve "a" is greater. Due to passage through the material, the amplitude of the longitudinal displacement will decrease to a certain extent, due to damping, and the wave front will be less steep.

Actual traveling time is represented by the distance between points R and S, which indicate the first infinitesimal displacement. Impulses transmitted from the pick-ups cannot attain sufficient force until the displacement has reached a certain value x_p. The reaction of the pick-up, P_I, is delayed t_a thereby. On arrival of the impulse at the pick-up, P_{II}, the delay is $t_a + \Delta t_a$, and the traveling time recorded is increased by Δt_a, because of the change of the wave front produced by damping ($x_{a,2} < x_{a,1}$). Hence the velocity observed will be lower than the theoretical velocity.

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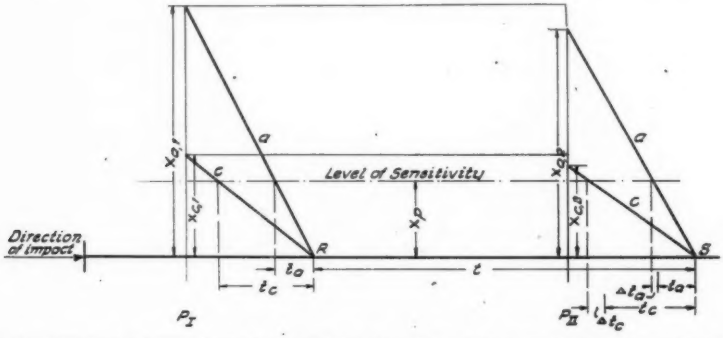

Fig. B—Steepness of wave fronts is changed by damping, causing an increase in traveling time the magnitude of which depends on the level of sensitivity and impact energy

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has been shown that the condenser chronograph measures longitudinal waves in the specimen, *i.e.*, waves spreading through the body contrary to Rayleigh waves which are only affecting particles at the surface of the body. In routine testing of beams, the blow is applied to the center of the end face, and it is believed that the wave spreads in all directions with varying velocities in aggregate and paste. Blows have been applied at different locations on the end face with no significant difference, the variations being less than 1 percent. It is believed that the pick-ups react to the first arriving displacement regardless of the path.

It has been found that in the case of a crack the results show a delay in transmission time for the pick-up position just behind the crack, but pick-up positions at distances greater than 20 cm have no influence in cases where the crack is only a part of the section. This shows that the wave may follow different paths through the concrete and not necessarily the surface. It is agreed, however, that the condenser chronograph is not able to analyze the conditions of concrete in different locations of a specimen as shown by Mr. Jones in Fig. A, which is a most interesting contribution to the discussion.

The authors are pleased to see that Mr. Jones has found a similar trend in the decrease of standard deviation when hardening progresses.

The procedure proposed by Mr. Jones of making auxiliary measurements on test cubes of the same concrete mixes as used in the structure is highly recommended, but in a number of cases where the purpose is to investigate older concrete members, it is only possible to obtain an impression of the relative quality of different members produced from the same mix at the same time.

The equipment used by Messrs. Leslie and Jones is based on high-frequency directional pulses, and experience shows that this equipment is suitable for measuring on structural members where the transmitter and receiver can be placed on opposite faces. The authors believe that this technique can be applied only with difficulty to large areas of adjoining concrete slabs, such as runways and wide roads, in which cases the condenser chronograph seems to be more suitable as the impulse can be started from a joint and registered through pick-ups on the surface.

The great difference in wave length of the produced pulses suggests that results obtained with different types of equipment will not necessarily be influenced in the same way by the hardening process, water and voids content, and Poisson's ratio. Hence further fundamental research on pulse propagation is still needed.

Title No. 48-41

Slip-Forms for Concrete Canal Lining*

By T. V. D. WOODFORD†

SYNOPSIS

Two types of slip-forms for placing concrete lining in irrigation canals are described. One type, which rides directly on the previously prepared subgrade, is a recent development for use primarily in small canals and farm ditches. Both types of equipment are now in wide use and have effectively reduced the cost of lining operations, fulfilling an urgent need for the conservation of water in the western states.

INTRODUCTION

Slip-forms for placing concrete lining in irrigation canals differ considerably from sliding forms (sometimes called slip-forms) used in the construction of concrete walls, bins, and grain elevators. Sliding forms used in building construction must usually be left in place long enough for the concrete to harden (initial set) while slip-forms for canal lining serve only to consolidate, screed, and finish the concrete as they move along longitudinally in the previously shaped earth section.

Two basic types of canal lining slip-forms are in present use. One, which is supported on, and guided by steel railroad rails placed along the top of both banks, has been in general use in large canals for about 16 years. The other type which rides directly on the subgrade without the use of guide rails, was brought into general use only recently for lining small canals and farm ditches. Use of the larger rail-mounted equipment appreciably reduced the cost of concrete lining in large canals, but the expense of setting and removing the rails is seldom justified in small canal sections where the volume of concrete per length of canal is proportionately small. The economies resulting from speedier construction and a reduction in manual labor are largely offset by the cost of guide rails as the size of the canal decreases. Yet there has been a growing need for dependable low cost linings to control seepage, weed growth, and erosion in thousands of miles of small canals and farm ditches in the western irrigation states. Faced with higher construction costs in recent years and the increasing urgency of water conservation, the Bureau of Reclamation, in cooperation with industry and other irrigation interests, has been active in a program of field and laboratory experimentation to lower the cost of canal linings. The development and successful use of subgrade-guided slip-forms was a direct result of these efforts.

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RAIL-MOUNTED SLIP-FORMS

Rail-mounted slip-form machines consist essentially of a framework traveling on rails at the top of each bank which supports a working platform at the bank level, a distributor plate (or drop chutes) extending from the working platform to a compartmented concrete supply trough with a vibrator tube in the bottom, and the slip-form. Although the whole rig is commonly referred to as a slip-form machine, the slip-form proper is a steel plate, curved up at the leading edge, extending across the bottom and up the side slopes of the canal to finish the lining over the entire perimeter in one pass. If a distributor plate is used, it is fastened to the leading edge of the slip-form and extends upward on a steep incline to the rear of the working platform. If equipped with drop chutes, these convey the concrete from the working platform to single compartments in the trough below. Concrete is usually dumped into the drop chutes or against the distributor plate from a shuttle car operating on the working platform. As the concrete passes out at the bottom of the trough and under the slip-form, it is consolidated by a vibrating tube located parallel to and a few inches ahead of the form. Diesel-electric generators, mounted on the most recent slip-form machines, supply power for their forward motion and for the many other motored operations thereon. Concrete for the lining is mixed and supplied to the slip-form machine by separate equipment which operates along the roadway at the top of either bank. Highway paving mixers, usually used for this purpose, are supplied with dry batches by truck from a central proportioning plant. On some of the large jobs, two mixers have been employed and belt conveyors have been used for delivering the concrete to the slip-form hopper. Rail-mounted slip-forms are adaptable to placing either plain or steel reinforced concrete lining of any practical thickness and in any size canal.

Equipment for final trimming of the subgrade, used in conjunction with rail-mounted slip-form machines, is supported on the same rails and is mechanized to a similar degree. Other rigs for supporting workmen and equipment for any hand finishing of contraction grooves and for the application of curing compound follow the slip-form and likewise travel on the same rails.

The first extensive use of rail-mounted slip-form machines, to the writer's knowledge, was in 1935 on lining operations of the Colorado River Aqueduct built by the Metropolitan Water District of Southern California. As stated by Lewis H. Tuthill,[1] the aqueduct contained a total of 62.2 miles of canal section lined with reinforced concrete. The lining in the 20 ft wide bottom was 8 in. thick and that on the side slopes decreased from 8 in. at the toe to 6 in. at the top. The sides sloped at $1\frac{1}{2}$:1, the water depth was 10.21 ft, and the lined perimeter, including freeboard was 62.22 ft. Mechanized rail-mounted trimming equipment and the slip-form machine both developed by the contractor on this job proved practical and economical. Similar equipment was used as early as 1937 for placing reinforeed concrete lining in the Roza Main Canal of the Bureau of Reclamation Yakima Project in Washington (Fig. 1). Since then rail-mounted slip-form machines have been

Fig. 1—Early rail-mounted slip-form placing reinforced concrete lining in Yakima Ridge (Roza Main) Canal, Washington

in common use, where the size of the job justified, in placing both plain and reinforced concrete lining.

One of the earliest unreinforced linings placed by rail-mounted slip-form on a Bureau of Reclamation project was in the Pasco Pump laterals of the Columbia Basin, Washington, in 1947. Being somewhat of an experiment in unreinforced lining, this installation contained approximately 26,000 sq yd of lining 3 in. thick and 15,000 sq yd 2 in. thick with lining perimeters of 19.8 and 17.3 ft, respectively. More recently, canals of unprecedented size in the Columbia Basin and in the Central Valley of California have been lined with concrete placed by rail-mounted slip-forms. The upper 95 miles of the Delta Mendota Canal, built by the Bureau of Reclamation in the Central Valley, is lined with unreinforced concrete 4 in. thick, has a constant bottom width of 48 ft, side slopes of 1½:1, and a vertical lining height varying from 15.54 to 17.87 ft (Fig. 2). Highly mechanized rail-mounted slip-form machines placed as much as 15,250 sq yd of this lining in 1185 linear ft of canal in a single 8½-hour day.[2]

SUBGRADE-GUIDED SLIP-FORMS

Slip-forms for concrete canal lining which ride directly on and are guided by the subgrade combine a forward skid plate shaped to fit the canal, a rear screed or troweling plate of the same shape, but offset from the skid plate by the desired lining thickness, and a middle hopper open to the subgrade bottom and side slopes. Concrete dumped into the hopper oozes out over the canal perimeter and is struck off, consolidated and troweled by the rear plate as the slip-form is dragged forward by one or more tractors or by a motor-driven winch mounted on the slip-form.

Such a device was patented by G. W. Gale in 1911 (United States Patent No. 992,649). In 1915 a subgrade-guided slip-form was developed and used

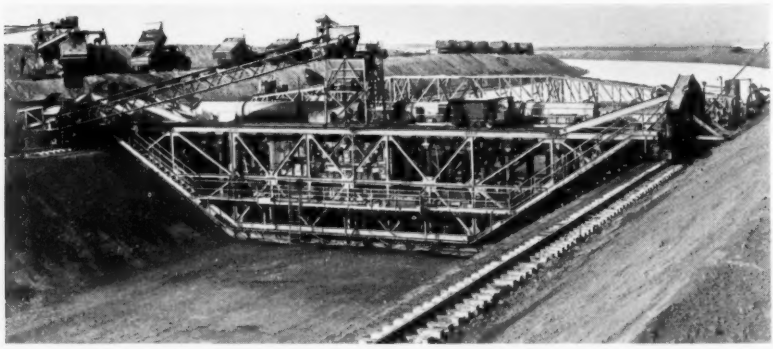

Fig. 2—Rail-mounted slip-form placing 4-in. unreinforced concrete lining in Delta-Mendota Canal, California. Note finished lining in background, batch trucks and paving mixer on top of bank and belt conveyor supplying concrete to center hopper

for placing concrete lining in canals of the Umatilla Project of the Bureau of Reclamation.[3] However, there is no record of the use of subgrade-guided slip-forms in the years that followed until Clyde E. Shields, engineer on the Bureau's Gila Project in Arizona, developed one of similar design in 1946. Probably the chief reason this type of equipment was not used in the intervening years is that most concrete linings constructed during this period contained steel reinforcement and the subgrade-guided slip-form is not adaptable for placing reinforced lining. The reinforcing steel placed in the section ahead of lining and supported at the proper distance from the subgrade interferes with operation of the slip-form which must ride directly on the subgrade. Furthermore, there was a continuing effort all through the thirties, when construction costs were low, to attain perfection in the surface and alignment of the finished lining. The rigid requirements of most specifications, therefore, were very difficult if not impossible to meet with subgrade-guided slip-forms. However, following World War II with rising costs for steel and better control of concrete mixing and placing operations, there was not as much justification for the use of reinforcement. After due consideration of these facts, Bureau of Reclamation design practices were changed to omit steel reinforcement in concrete canal lining except in special cases as when safety is involved. This change together with a reasonable liberalization of specification requirements for line, grade and finish made possible and actually encouraged the development and use of subgrade-guided equipment.

The slip-form developed by Shields was built to place a lining 2 in. thick having a rounded bottom and sides sloping up at 1½ horizontal to 1 vertical. This shape was adopted because of the availability of an old dredge which was modified and used to do the final trimming of the subgrade. The dredge cut a section with 1½ to 1 side slopes and a bottom rounded to a 30-in. radius. The front skid plate of the slip-form was shaped to fit this trimmed section

Fig. 3—Shields subgrade-guided slip-form placing 2-in. unreinforced lining on Bureau of Reclamation project near Yuma, Arizona. Truck-mounted mixer supplying concrete to slip-form as both move forward

and the rear troweling plate, with a bottom rounded to a 28-in. radius, controlled the lining thickness of 2 in. over the entire section. To provide a 4-in. curb at the top of the lining on each side slope, special brackets were attached to the forward and rear plates, one to notch the bank and the other to form the curb. These brackets were movable along the underside of the two plates to adjust for different canal depths (and corresponding carrying capacities) up to a maximum of 4 ft. Skid shoes attached to both sides of the slip-form at the top of the canal bank were used to support the form off the subgrade and to skid it forward for cleaning at the end of each day's operation. However, skid shoes are not essential if some type of hoisting equipment is available for lifting the slip-form.

Most of the concrete placed with the Shields slip-form during the first year of its operation was mixed in a one-half cubic yard mixer mounted on the flat-bed of a truck (Fig. 3). This put the mixer high enough that the concrete dumped into a trough was thus conveyed into the slip-form hopper. A tank wagon, which supplied mixing water, and the truck-mounted mixer, which was supplied with dry batches from auxiliary trucks, both moved along the canal bank with the slip-form. This rather inefficient arrangement for supplying the slip-form with concrete limited the rate at which lining was placed to less than 700 sq yd per day in canals having a lining perimeter of about 11 ft. About three times this rate of placement was attained with the experimental use of a traveling plant mixer for supplying the slip-form

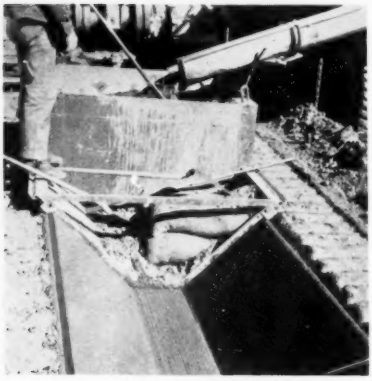

Fig. 4—Small subgrade-guided slip-form placing 1½-in. unreinforced lining in farmer's ditch near Phoenix, Arizona

Photo courtesy Portland Cement Assn.

with concrete. This equipment picked up the proportioned dry cement and aggregate from a window placed along the canal bank, mixed it in a pug mill type mixer, where water was added, and dumped it by conveyor belt into the slip-form hopper.

The same type equipment was used for mixing and supplying plastic soil-cement to a subgrade-guided slip-form which placed about three-fourths mile of experimental lining in a lateral of the W. C. Austin Project (Bureau of Reclamation), Oklahoma, in May 1947. The slip-form was built by the Madsen Iron Works of Huntington Park, Calif., through arrangements made by the Portland Cement Assn. After some necessary on-the-job modifications, this slip-form did not differ much from the Shields equipment except that it was shaped in the more conventional trapezoidal canal section with a flat bottom. Soil-cement for canal lining was believed to have merit for use in localities where satisfactory aggregate for concrete is not economically available and the sandy soil of the W. C. Austin Project was well suited for a trial installation. The plastic soil-cement was efficiently placed by slip-form at relatively low cost and satisfactorily reduced seepage losses.

As news of the successful use of these simple subgrade-guided slip-forms spread, Bureau of Reclamation forces on other projects and contractors on both public and private work built similar equipment for use in lining small canals and farm ditches. One contractor in the Phoenix, Ariz., area, after the successful operation of a small slip-form which he built for his own use, built additional units for the use of other contractors on a rental basis. A brief article on the Fullerform, as it was called, appeared in the March 1948 issue of *Western Construction News*. The Fullerform, first put into service in December 1947, had a flat bottom 12 in. wide with sides sloping at 1:1 and a concrete hopper divided so that concrete was placed first on the two side slopes and then on the bottom (Fig. 4 and 5). The rear troweling plate was offset only 1½ inches from the front skid plate to form a lining of the same thickness.

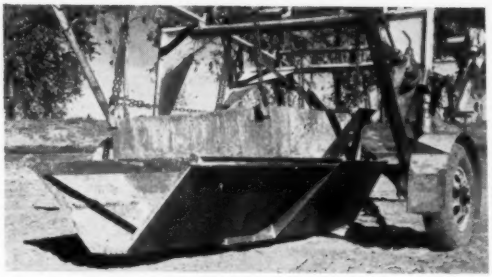

Fig. 5—Fullerform suspended from 2-wheel trailer for transporting. Note opening in side form and offset (1½ in.) of rear screeding plate

Photo courtesy Portland Cement Assn.

Concrete for the most of these small slip-forms has been mixed and supplied by transit truck mixers but in some instances where air-entraining concrete was used, batches have been hauled in dump trucks from a central mixing plant. On at least one job, concrete was mixed in a highway paving mixer which operated along side the canal and was supplied with dry batches from a central proportioning plant. It is not unusual for small subgrade-guided slip-forms in combination with adequate mixing equipment, to place a mile or more of 1½-in. unreinforced lining in 1 day in ditches with lining perimeters of 5 to 6 ft. Short stretches of this size ditch have actually been lined at a rate of about 1 mile per hour, demonstrating the possibilities if sufficient concrete can be made continuously available.

An adjustable plow-type ditcher has proven satisfactory in most localities for excavating and final shaping of the earth section of these small canals. The ditcher is pulled by one or two tractors operating in the bottom, straddle the ditch, or on both banks depending on the size of the section. In new excavation, several passes of the ditcher are required to remove the earth and properly shape the section for lining.

The maximum size of canal which it is possible to line with a subgrade-guided slip-form has not been definitely established. Such equipment has been satisfactorily used in a canal with lining perimeter of 17.6 ft and it is believed that this is not far from the upper practical limit.

SUMMARY

Development of the subgrade-guided slip-form has made it economically feasible to line small canals and farm ditches with concrete, thus aiding in the conservation of irrigation water so vital to agriculture in 17 western states. The extent of the present use of such equipment is indicated by a conservative estimate that more than 300 miles of concrete lining was placed by subgrade-guided slip-forms in canals and farm ditches in the state of Arizona alone during 1951.

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Discussion of a paper by T. V. D. Woodford:

Slip-Forms for Concrete Canal Lining*

By LEWIS H. TUTHILL and E. C. WENGER

By LEWIS H. TUTHILL†

The author has given an excellent summary of the nature of this equipment and of the character of work it will do. From his mention of the 300 miles of lining laid by the subgrade-guided slip-form, it may be inferred that rail-guided machines have had a more limited use. Actually they have placed concrete lining in at least 640 miles of canals including many of unusual size. The total of ditch lining will doubtless some day surpass this figure, but its size will remain impressive and indicative of the "know-how" that has been attained by builders of the equipment and by many contractors who have operated it.

The significance of the rail-guided slip-form in canal lining work suggests that perhaps a few supplemental details would be worthy of record. The prime requisite for successful operation is ample vibration of the concrete as it feeds under the forward edge of the slip-form. For this purpose, an adequately vibrating tube parallel to and operating a few inches from the leading edge of the slip-form has been found to lay a completely consolidated lining, while all attempts to vibrate the slip-form plate have met with failure. This failure occurs because once concrete is under the leading edge of the slip-form without being consolidated and filled to full thickness of the lining, no amount or kind of vibration of the slip-form plate can add the concrete necessary to fill unfilled areas of lining, either on the bottom or side slopes. The concrete must be vibrated so that it will mold into a full, consolidated slab just as it goes under the slip-form.

Uniformity of correct concrete mix and slump is a vital factor in attaining high speed and good results. When vibration is what it should be, air-entraining concrete of 2-in. slump is most satisfactory. This consistency avoids sagging and highlighting of longitudinal reinforcing rods where used on side slopes, and a sloppy surface for finishing the bottom. When vibration is attuned to effective handling of 2-in. slump concrete, variations causing slumps less than 1½ in. are likely to cause ragged unfilled areas in the lining. Finish screening of coarse aggregate at the batching plant and use of well-drained sand has contributed materially to slump uniformity. Although

*ACI Journal, Apr. 1952, *Proc.* V. 48, p. 637. Disc. 48-41 is a part of copyrighted Journal of the American Concrete Institute, V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

†Concrete Engineer, Bureau of Reclamation, Denver, Colo.

batch trucks supplying mixers at the lining machine is the set-up surest to produce uniform concrete, truck mixers and agitators have given no difficulty in this regard because of the speed and large concrete demand of these lining machines.

Smoothest operation, best finish, and better dummy joint grooves result when the amount of sand in the mix is increased by a factor of about 1.10 over the amount which would be optimum for ordinary concrete work.

In recent years, improvement in operating details of the machines, in vibration, and in factors making possible the production of more uniform concrete have resulted in a much faster and thus a less expensive operation on the unit basis. These improvements have produced in many cases a lining surface as it emerged from the slip-form that was fully satisfactory without further hand troweling and finishing. Some operators further improve and assure their finish by use of "ironer" plates under spring pressure which follow immediately behind the slip-forms.

By E. C. WENGER*

Mr. Woodford describes in an excellent manner the equipment currently used for lining irrigation canals. The value of concrete-lined canals in reducing water loss as well as operation and maintenance costs has long been known. This paper quite correctly credits the recent development of subgrade-guided slip-forms in reducing lining costs for laterals and farm ditches.

Another cost reducing factor is the development of ditching equipment

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Fig. A—One type of ditching equipment used in excavating a 7-ft bottom irrigation ditch

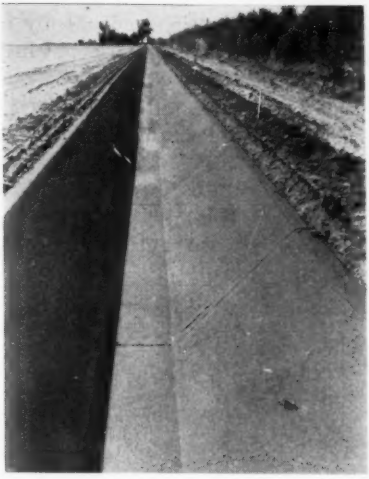

Fig. B—Farm irrigation ditch lined with concrete by subgrade-guided slip-form. Note tooled contraction joints at 15-ft intervals and special farm ditch outlets

which permits rapid excavation of the canal to dimensions which are within practical limits for efficient operation of the subgrade-guided slip-form (Fig. A). These important developments have been largely responsible for bringing concrete linings within the economic reach of irrigation districts and individual land owners.

Because of the increased and widespread use of subgrade-guided slip-forms for the smaller canals (Fig. B), the importance of uniform compacted subgrades should not be overlooked. On the larger canals where rail-mounted slip-forms are used, compaction of the subgrade is usually specified and obtained during construction. On the smaller canals where subgrade-guided slip-forms are used, this provision occasionally has been neglected, resulting in settlement and damage to the lining.

Proper curing of the concrete also is important in obtaining full value of the lining. Curing can be done by ponding, with wetted earth or straw cover, or by application of a liquid membrane sealing compound. The latter is being increasingly favored since it can be applied rapidly and little labor and equipment are needed.

The use of pit-run aggregate in areas where satisfactory deposits are available also reduces the cost. Cost of screening and recombining aggregates is high when compared to the use of aggregates as they are found in the pit.

The importance of using air-entraining concrete should be mentioned. The advantages of air entrainment are better workability with less water and lower sand content, increased cohesiveness, less segregation and bleeding, and better durability in freezing climates.

While vibration is used effectively in placing concrete with the rail-mounted slip-form it has not been used to any great extent with the subgrade-guided

slip-form. Vibration undoubtedly would permit satisfactory placement of a lower slump concrete and might increase the efficiency in placing sufficiently to pay for the added cost.

Of interest are some of the recent larger projects on which subgrade-guided slip-forms have been used. These are: (1) Yuma Mesa Division, Gila project, Ariz., 24,775 sq yd in 1947; (2) North Platte project, Neb., 30,000 sq yd in 1950; (3) Columbia Basin project, Wash., 48,400 sq yd under construction in 1952; and (4) Mohawk distribution system in the Gila project, Ariz., 305,000 sq yd, of which 211,000 sq yd are now under contract.

Short Cuts in the Design of Continuous Structures*

By I. E. MORRIS†

SYNOPSIS

In preliminary design to determine concrete member sizes or cost comparisons, time-saving methods involving a reasonable degree of accuracy are desirable. Moment coefficients for both positive and negative moments are limited to spans which are approximately equal. In the proposed method, coefficients are used for negative moments only. With known negative moments, positive moments are easily determined by the application of simple statics. Whether the method has the requisite accuracy for a final design depends largely upon the implicitness of the designer's faith in elastic stress analysis.

PRELIMINARY CONSIDERATIONS

For preliminary design, or when comparative cost data on different schemes of framing are sought, it is well to be able to obtain such data as expeditiously as possible. The architect, in order to develop his details, must know the size of beams and columns and the thickness of slabs. To supply this information, the structural engineer must do some designing, unless he is proficient at guessing the size of members. If he proceeds with what may be termed a final design, using moment distribution, or some other form of elastic analysis, he is likely to run into a duplication of work, for architects' preliminary sketches frequently undergo considerable change during the preparation of working drawings. A change in the position of a column, or the interruption of a series of beams, often nullifies pages of computations. Therefore, a great deal of time and effort are saved when, by some approximate method, member sizes and steel tonnages may be established which will hold for the final design. The method here presented is not intended as a subtstiute for any form of analysis, except in the initial stages of the design; nor is it intended to comply in every respect with the theory of limit design, although it no doubt closely approaches that theory.

In structural engineering, more so perhaps than in any other field of endeavor, sound judgment and experience are invaluable assets. If a designer cannot visualize the action of a loaded structure, his task is almost hopeless. He is merely floundering in a mathematical miasma and is never certain that his design has achieved maximum effectiveness. Such a designer is never in complete command of his structure.

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ASSIGNMENT OF MOMENTS

In the analysis of continuous beams, we provide for a total moment equal to, or slightly greater than, the simple beam moment of the loaded beam. We may assign portions of this moment to any points where it can be safely resisted, but in every case the total simple beam moment, at least, must be accounted for. This is illustrated in Fig. 1 (A), (B) and (C).

Here we have in (B) the simple beam moment curve, the negative moment curve, and the combined moment curve, in this case the maximum negative moment being equal to the simple beam positive moment, or $WL/8$. Note also that the construction ordinate 2'-6 at the span centerline is $WL/8$. It may appear at first glance that the maximum positive moment could be

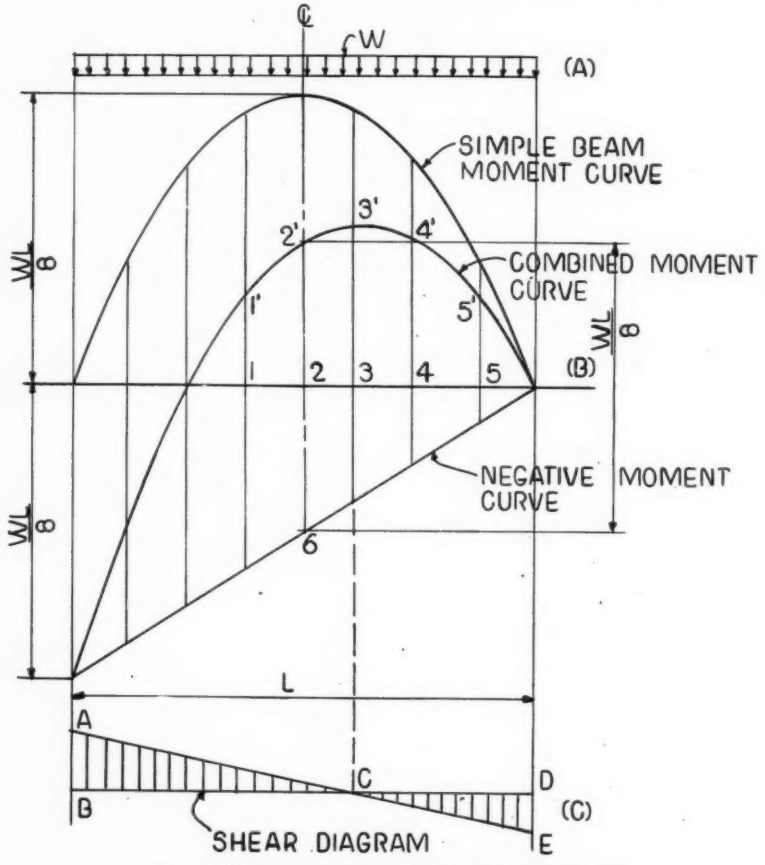


Fig. 1—Moment curves for uniformly loaded simple beam

obtained by subtracting the negative moment ordinate 2-6, which is obviously $WL/16$, from the maximum simple beam moment; but this would not be correct, since the ordinate 3-3′ is longer than 2-2′. However, if we calculate the shear at each end of the beam, and then compute the positive moment, we will obtain a value equal to the ordinate 3-3′, if the latter is drawn properly to scale. Moreover, the diagram shows that the sum of the ordinates 2-6 and 3-3′ is slightly greater than $WL/8$.

The ordinate 3-3′ is the maximum positive moment in the span. And this ordinate always falls directly over the point of zero shear, as shown in the shear diagram, Fig. 1 (C). Since the shears in the beam are affected by the end moments, the greater shear being at the end having the greater negative moment, it follows that the point of zero shear will move to the right, or to the left, in accordance with the variation in negative moments. In short, this zero point follows the maximum construction ordinate of the positive moment curve.

Another interesting relationship is that, in Fig. 1 (C) the area of the triangle ACB minus the maximum negative moment ordinate $WL/8$ is equal to the positive moment ordinate 3-3′. Also, the area of the triangle DCE is equal to this ordinate.

In case the end moments are equal, as in Fig. 2 (B), the maximum positive moment does occur at the center of the span. And here the positive moment is $WL/8$ minus the negative moment. The point of zero shear (not shown) is also at the center of the span.

Fig. 3 shows a condition in which the negative moments at the ends of the beam are not the same. But again the construction ordinate at the center of the span is equal to $WL/8$. And the maximum positive moment is not at the center of the span, although it is nearer the center than it is in Fig. 1 (B), for the reason that the shears at each end of the beam are more nearly equal. In every condition, though, we are dealing with the fundamental equation $M = WL/8$.

It would seem, after a study of Fig. 2, that we could continue to build

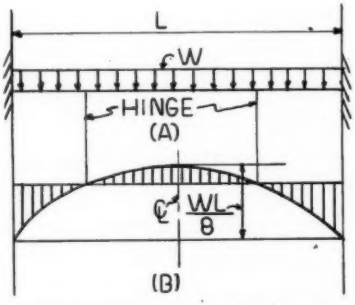


Fig. 2—Maximum moment in fixed end beam with equal end moments

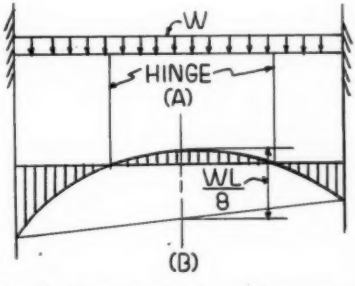


Fig. 3—Unequal negative end moments in beam

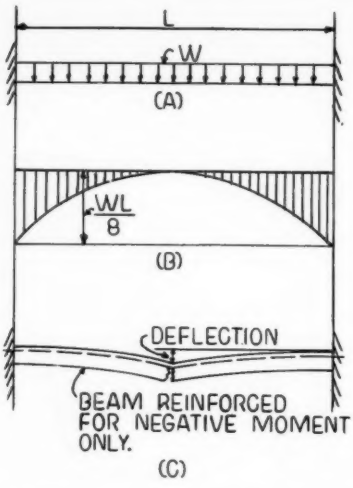


Fig. 4—Moments in beam reinforced for negative moment only

up negative moment at the ends of a beam till the positive moment would be zero. In fact, we can, and thereby produce two cantilevers, provided there is sufficient restraint at the ends to resist the moment, and provided the beam is either cut at the center (as in Fig. 4 (C)) or is held by joints each of which rotates so as to bow the entire beam upward. This condition is shown in Fig. 4 (B).

The foregoing seems to indicate that in a series of continuous beams, regardless of span ratios, an experienced designer may, with sufficient accuracy for all practical purposes, assign negative moment at the supports and, with these moments, proceed with the calculation of positive moments. This procedure would no doubt introduce a situation in which two designers, designing the same beam, would come up with different answers, both of them correct, since each would provide for a total moment of slightly more than $WL/8$.

Suppose we consider the condition in Fig. 5 and assume all beams to be the same size, a size ample for shear based on approximate reactions. Such shears are easily obtained by adding to the static shears 10 or 15 percent to provide for continuity.

The K value of the beam on the 10 ft 0 in. span is twice that of the beam on the 20 ft 0 in. span. This means that the short beam would have twice the resistance to rotation that the long beam would have. Actually, however, the capacity of the short beam is dependent, in part at least, upon the

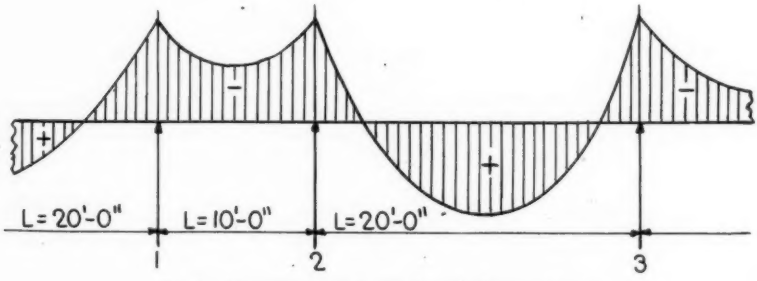


Fig. 5—Moments in continuous beams of unequal span

amount of reinforcement used, if we agree that $M = A_s f_s j d$. If the beam in the short span were of lesser depth than the one in the long span, the negative moment at the supports, by moment distribution, would be less than it would be according to our original assumption, which was that all beams are the same size. But, in case of a shallower beam on the short span, we can *assume* a smaller moment at the supports and increase the positive moment in the long span, which is in agreement with the relations illustrated in Fig. 1 to 4. The beam sections obtained from these moments would hold for the final design, and the amount of reinforcing steel would be close enough for an estimate. Therefore, instead of figuring stiffnesses and distributing moments for a preliminary design, we may assign a *reasonable* negative moment to the supports and proceed from that point.

COMBINATIONS OF SPANS

The combination of spans shown in Fig. 6 involves a bit more visualization.

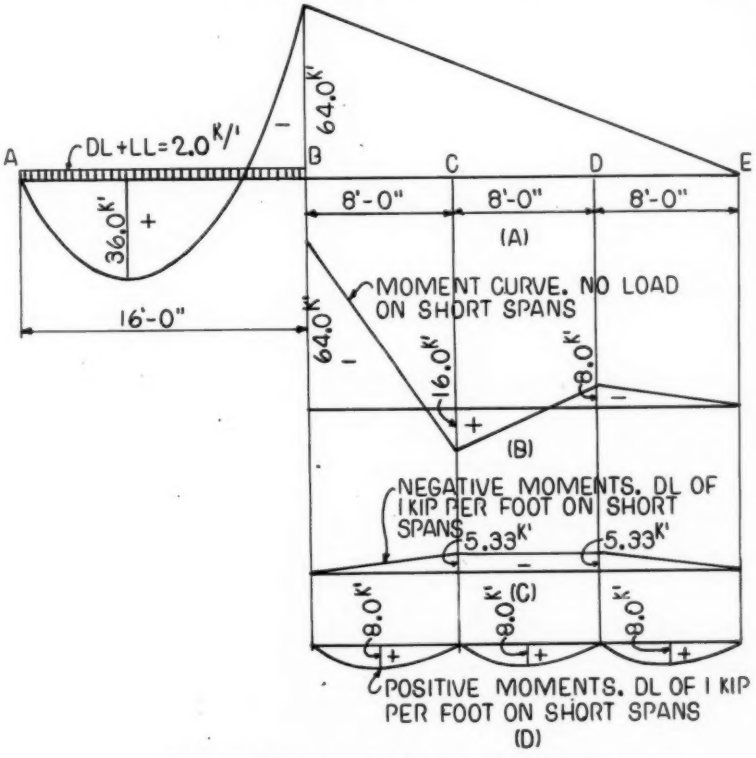


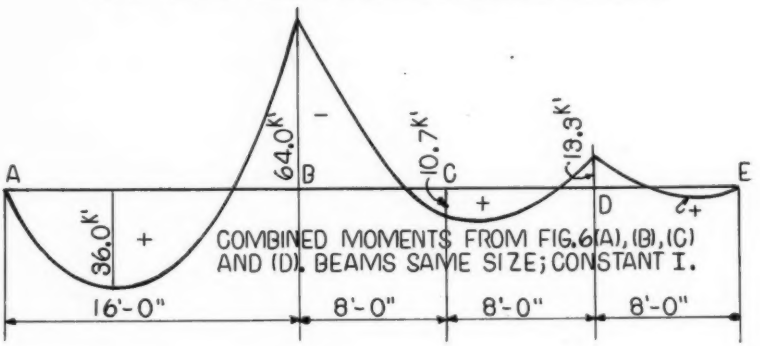
Fig. 6—Conditions in unequally loaded unequal spans

Here full dead and live loads are on span AB, with no load on the other three spans. Unless the beam were anchored at supports C and D (anchorage is assumed at E) the moment at support B would lift the beam off the supports at C and D. If we apply at C a downward force sufficient to pull the beam down to the support at that point, it is obvious that tension would occur at the underside of the beam.

Let us assume that, when the beams are forced back to their normal position, there is at C a moment equal to one-fourth the moment at B, and at D a moment equal to one-eighth the moment at B. This condition is shown in Fig. 6 (B). Now, suppose we place on the short spans a dead load equal to one-half the load on AB. Assigned negative moments of $WL/12$ are shown in (C), and the simple beam moments in (D). If all these moments are added algebraically, the curve of combined moments is as shown in Fig. 7.

Fig. 8 shows moments obtained by moment distribution. Note that by the latter method the negative moment at B is very nearly $WL/10$, L being the long span. This coefficient is the one which would normally be employed in calculating the negative moment at B. In this instance $WL/8$ was used to illustrate an extreme condition.

Fig. 7—Combined moment curve obtained by negative moment method



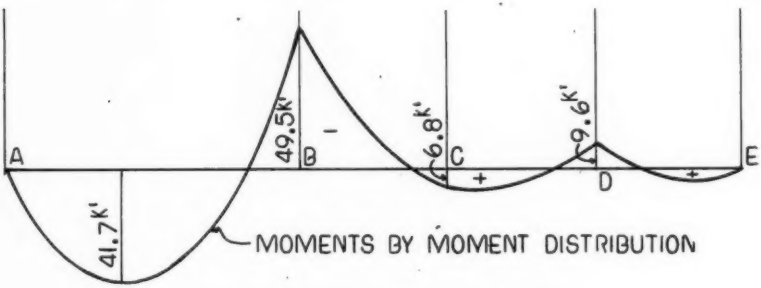


Fig. 8—Combined moments by moment distribution

The question may now arise regarding the procedure when the three short spans are not equal and are not one-half of the long span. Assume span BC to be 6 ft 0 in., CD 10 ft 0 in., and DE 8 ft 0 in. We may again assign to supports C and D respectively ¼ and ⅛ of the negative moment at B. But in calculating the dead load negative moments at supports C and D, by use of the coefficient $WL/12$, L should be 10 ft 0 in. for the moment at C, and 8 ft 0 in. for the moment at D. This is because the longer span CD reduces the positive moment at C.

It will be noted that in Fig. 7 negative moments at supports B and D, and the positive moment at support C, are greater than those at corresponding supports in Fig. 8. But the positive moment in span AB of Fig. 8 is greater than the corresponding moment in Fig. 7; so that, when we consider economy of steel, there would be very little difference, and the final design would be largely a redistribution of the reinforcement.

If the beam BC were considerably shallower than AB, we could place over support B an amount of top steel which would develop the compressive strength of the smaller beam. The resisting moment thus obtained would be the negative moment to use over support B. This would, of course, increase the positive moment in span AB. Negative moments at supports C and D could then be assumed as $WL/12$ (or any other reasonable coefficient), and it would probably be found that the new conditions would produce no positive moment at C. But there should be *some* steel in the bottom of the beam at this point, and the minimum requirement would safely resist any positive moment which might develop. Here it should be observed that, with beam BC smaller than beam AB, no analysis, no matter how theoretical, could make this small beam develop more moment than it is capable of developing. Here we have the possible situation of the small beam definitely limited in both width and depth, in which case we would either have to assign to it a negative moment it could resist, or treat span BC as a simple span, or admit that structural theory does not apply to such a condition. Obviously, to admit the latter would be unrealistic.

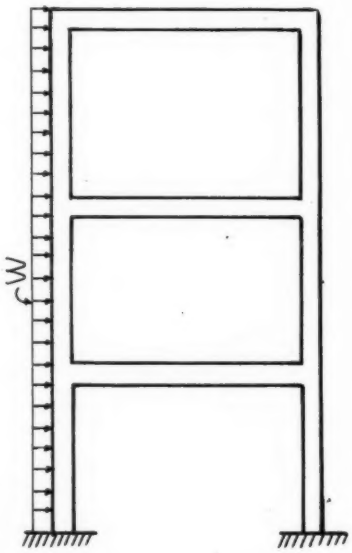


Fig. 9—Wind load on two-column bent

WIND STRESSES

Calculation of wind stresses in reinforced concrete frames is another operation in which a lot of preliminary effort is wasted. Fig. 9 shows a two-column bent which must resist wind load. Suppose we assume that this bent has been adequately designed. There are moments and shears in all members. Now, suppose we add a bay on each side of this bent and produce a four-column bent under the same load as shown in Fig. 10. Can we reasonably say that the four-column bent, without any further distribution of moments, will not safely resist the wind load? If we conclude that the four-column bent is safe, does it not mean that a concrete frame will resist forces wherever there is strength to resist them? It is granted, of course, that the strength, or the capacity to resist moments and shears, must be distributed in some logical manner throughout the frame.

There will be some rotation of all joints, however slight. But there will also be steel in all columns and beams resulting from vertical loads. If this steel has been proportioned on the basis of the most unfavorable position of live load, the moments in columns are not likely to be excessive; for, if the columns are designed for full dead load plus live load with permissible reductions, they will have excessive strength when live loads are in the most unfavorable positions for beam moments, because at such times all live load is not on all floors simultaneously. Therefore, to provide for the slight rotation of joints, we may take the sum of the moments in the 8 joints of Fig. 9, divide it equally among the 16 joints in Fig. 10, and reinforce the members accordingly. (Here the fixed ends of columns are considered as joints.) If greater accuracy is desired, the final design would result mainly in a redistribution of reinforcement.

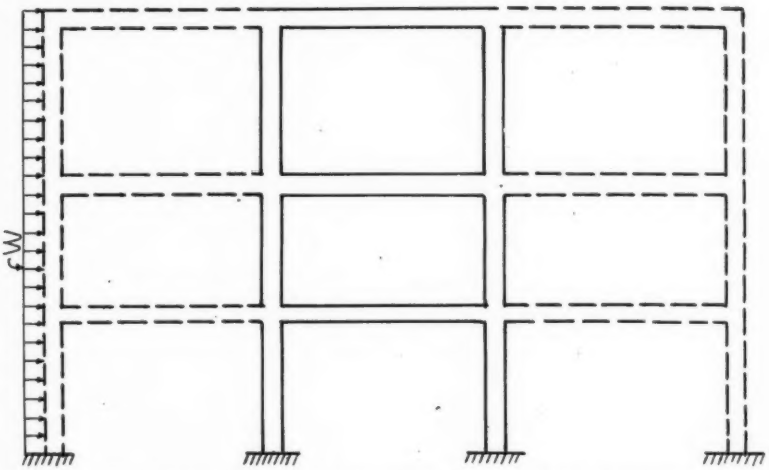


Fig. 10—Four-column bent formed by addition of a bay to each side of structure in Fig. 9

Illinois Experimental Continuously Reinforced Concrete Pavement after Four Years*

By J. D. LINDSAY† and H. W. RUSSELL‡

SYNOPSIS

Construction and design features of the experimental continuously reinforced concrete pavement placed in Illinois in 1947 are described. The performance of the pavement up to an age of about four years is assessed with stresses in the longitudinal steel, frequency and width of transverse cracks, localized failures and general condition of the pavement being discussed. Economic possibilities of this type of pavement are not considered because of insufficient service life.

INTRODUCTION

Continuous reinforcement in concrete pavements is not a new idea. It was used as early as 1921 in the Columbia Pike experimental pavement near Washington, D. C. Outstanding examples of this type of construction since then are the Indiana experimental pavement, slope pavement at the Lake Mathews Reservoir in California and the canal lining in the aqueduct of the Metropolitan Water District of Southern California.

Interest in this type of construction was revived recently largely through the efforts of W. R. Woolley, Bureau of Public Roads, through his paper "Continuous Reinforced Concrete Pavements Without Joints," which first appeared in 1946 and was later published in Highway Research Board *Proceedings*, 1947. In 1946 the Illinois Division of Highways decided to conduct an investigation of continuously reinforced concrete pavements, and in 1947, in cooperation with the Bureau of Public Roads, construction was started on the first of three proposed experimental pavements. Also in 1947, the New Jersey State Highway Department constructed two sections of pavement each approximately one mile long, having uniform thicknesses of 8 and 10 in. and containing 0.90 and 0.72 percent of longitudinal steel, respectively. In 1949, the California Division of Highways constructed a test section 1 mile long, 8 in. thick, with 0.5 percent of longitudinal steel in half the length and 0.63 percent in the remainder.

EXPERIMENTAL PAVEMENT

The experimental pavement described in this paper is officially identified

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as F. A. Route 12, Section 0-2, Fayette County. It is just west of Vandalia on U. S. Route 40, which carries a large volume of traffic. It is estimated from origin and destination studies made in the vicinity of the project that the pavement carries 2250 vehicles per 24 hr., consisting of 1700 passenger cars and 550 commercial vehicles. Construction was started in September 1947 and completed in May 1948.

The experimental pavement is 22 ft wide and 29,493 ft or about 5½ miles long, and is divided into eight test sections, six of which are approximately 3500 ft long and two are about 4230 ft. The pavement contains no transverse joints except those separating the test sections and construction joints at the end of each day's run. The reinforcing steel was carried across each construction joint so that it is continuous from end to end of each test section. The transverse reinforcing bars extended the full width of the pavement, creating continuous reinforcement in that direction also, and the customary center joint was omitted. The variables between test sections are thickness of pavement and amount of longitudinal reinforcing steel.

Four of the sections are uniformly 7 in. thick and four have a uniform thickness of 8 in. These thicknesses, chosen somewhat arbitrarily, were thought most likely to give the desired performance within economic limits.

The pavement was constructed directly over the natural soil as graded, without granular replacement or special subgrade treatment of any kind. About 90 percent of the pavement is on soils susceptible to pumping.

Previous experiments indicated that 0.5 percent of longitudinal steel may be sufficient for proper performance, although the theoretical amount is about 1 percent. For this project four percentages of longitudinal steel were used with each thickness of pavement; namely, 0.3, 0.5, 0.7 and 1 percent. These percentages, it was believed, provided sufficient range to establish the proper amount of reinforcement necessary to induce closely spaced transverse cracks and to keep them permanently closed under the action of the elements and heavy traffic. The distribution of the reinforcing steel in the various test sections is given in Table 1. Fig. 1 shows the assembled reinforcement in one of the sections.

PERFORMANCE OF PAVEMENT

The pavement has been under close observation since its construction and much data have been collected. Since the average age of the test sections is less than four years, no definite predictions can be made as to their ultimate performance. The data, however, show some highly significant tendencies and trends, which are of interest and worthy of consideration.

Stresses in longitudinal steel

Steel stresses are perhaps of greatest immediate significance because they give the earliest indication of the probable ability of the pavement to perform satisfactorily. Successful performance cannot be expected unless the stress in the reinforcing bars remains below the yield point of the steel. If

Fig. 1—Completely assembled reinforcement

the stress exceeds the yield point, the bars will stretch, the cracks will become progressively wider and eventually the steel may break. Since it is possible that near maximum steel stresses may occur during the first winter, especially if the pavement is built during high temperatures, stress measurements can be of great significance.

Stress was determined by means of SR-4 gages mounted on selected longitudinal bars in both the 7- and 8-in. sections which contain 0.7 percent of steel. The gages in both sections remained reliable only through the first winter following construction. The average maximum stress observed during that period in Section 3 (7-in. pavement, 0.7 percent steel) was 62,400 psi and that for Section 6 (8-in. pavement, 0.7 percent steel) was 42,000 psi. The yield point of the steel was approximately 69,000 psi.

When these stresses were observed, Section 3 had been subjected to a seasonal decrease in slab temperature of approximately 70 F, and Section 6 to only 45 F, which may be the principal reason for the considerable difference between the stresses for the two sections. It is also interesting to note that the stress away from the joint was never higher than 10,000 psi, which indicates that as long as the concrete is structurally continuous it will carry a large part of the total stress.

TABLE 1—DETAILS OF REINFORCING STEEL IN THE VARIOUS TEST SECTIONS*

Section	Length, ft	Thickness, in.	Longitudinal steel	
			Size and spacing	Percent
1	3504	7	$\frac{3}{4}$ in. at $5\frac{1}{4}$ in.	0.3
2	3504	7	$\frac{1}{2}$ in. at $5\frac{1}{16}$ in.	0.5
3	3504	7	$\frac{5}{8}$ in. at $6\frac{1}{4}$ in.	0.7
4	3504	8	$\frac{3}{4}$ in. at $4\frac{7}{8}$ in.	0.3
5	3504	8	$\frac{1}{2}$ in. at $4\frac{13}{16}$ in.	0.5
6	3508	8	$\frac{5}{8}$ in. at $5\frac{1}{16}$ in.	0.7
7	4233	8	$\frac{3}{4}$ in. at $5\frac{1}{16}$ in.	1.0
8	4233	7	$\frac{3}{4}$ in. at $6\frac{1}{4}$ in.	1.0

*Transverse reinforcement consists of $\frac{5}{8}$-in. round deformed bars spaced at 12-in. centers in half of each section and at 18-in. centers in the other half.

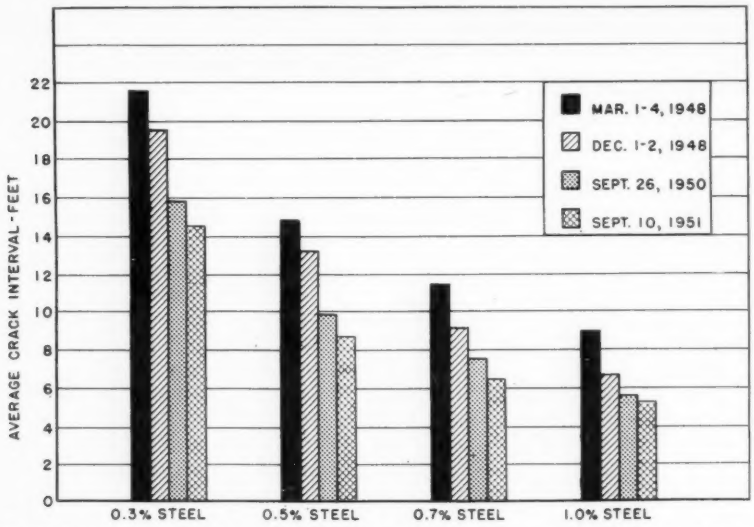

Fig. 2—Effect of amount of longitudinal reinforcing steel on average transverse crack interval —7-in. sections

Frequency of transverse cracks

The theory of continuous reinforcement assumes that frequent transverse cracks will develop in the pavement. The behavior of all the test sections has been according to theory in this respect, numerous transverse cracks having developed early in each section and new cracks are developing continually.

Fig. 2 shows the results of four surveys made to determine the average interval between transverse cracks in the four sections of 7-in. pavement. The most significant thing about these data is the very obvious relationship between crack interval and the amount of longitudinal reinforcing steel. It is clearly evident that the average crack interval increased as the amount of steel decreased, or the frequency of cracks increased with the amount of steel. The corresponding data for the 8-in. sections are very similar to those for the 7-in. sections. Fig. 2 also shows there was a decrease in the crack interval, or an increase in the number of cracks, with each successive survey.

Width of transverse cracks

Of equal importance, and closely related to steel stress, is the width of transverse cracks which develop in a continuously reinforced pavement. For effective performance the cracks must be narrow, preferably so small that they will not carry appreciable amounts of water to the subgrade. Crack width is a function of steel stress and the effectiveness of the bond between concrete and steel in the immediate vicinity of the crack.

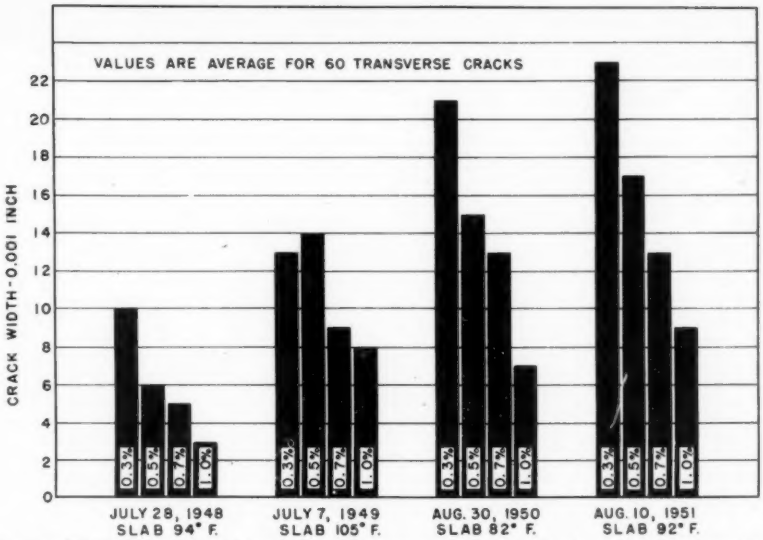

Fig. 3—Effect of amount of longitudinal reinforcing steel on width of representative cracks—7-in. sections

A comparison of the width of cracks is shown by Fig. 3 in which the average surface width of 60 representative cracks, measured with a microscope, is plotted for each of the four sections of 7-in. pavement. The chart shows that, in general, average crack width varies inversely with the amount of longitudinal steel, the cracks being widest in the 0.3 percent section and becoming progressively narrower as the percentage of steel increases.

A study of measurements taken at different periods of the year shows that crack width varies seasonally, cracks generally being relatively wider in winter, when the pavement is contracted and the steel presumably is under greater tension, than in the summer, when the pavement is expanded. However, a matter of some concern is the fact that the representative cracks are becoming progressively wider with age and do not close tightly in the summer when the pavement expands, but appear to be remaining further open with each successive summer.

This condition is also reflected by a progressive growth in the lengths of the various test sections and here also the effect of the steel is apparent, the growth being less for the sections with the heavier steel.

Physical condition of pavement

Generally speaking, the condition of the pavement is excellent and compares favorably with that of the adjacent 4½ miles of uniform 10-in. standard pavement constructed as a part of the same contract. The standard pavement has a 6-in. granular sub-base, 78-lb wire fabric reinforcement, contraction joints at 100-ft intervals and metal center joint with tie bars.

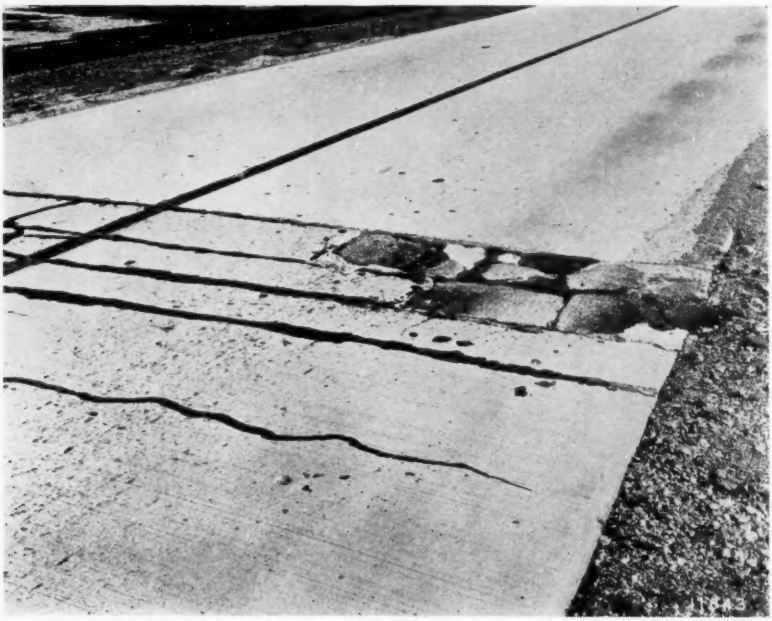

Fig. 4—Looking east from Station 292+75 showing excellent pavement beyond localized failure

Transverse cracks in the experimental pavement are for the most part invisible to motorists, even when traveling at moderate speeds. The pavement surface is smooth and has unusually good riding qualities. There is none of the rhythmic thud and pitching motion often experienced on pavements with closely spaced joints.

Since the pavement was built without a center joint, longitudinal cracking has developed, as expected. This cracking naturally does not coincide with the centerline of the pavement, but follows an irregular path, meandering as much as 3 ft on either side of the centerline. At several locations there are two approximately parallel longitudinal cracks, and at one place there are three. These cracks are noticeable only when one drives slowly. There is no apparent difference in longitudinal cracking where the ⅜-in. transverse bars are spaced 12 in. on centers and where they are 18 in. on centers. It does not appear that the longitudinal crack will be objectionable unless its width increases or it develops unsightly spalling.

Some failures were expected, since, to establish the minimum steel for proper performance, two sections were purposely designed with less steel than previous experience indicated might be adequate. Only two serious structural failures have occurred in the pavement so far, both being located

Fig. 5—Concrete patch placed at failed area shown in Fig. 4

in the 8-in. test section containing 0.3 percent steel. They were highly localized and involved relatively small areas of pavement.

The larger of the two occurred adjacent to a construction joint, about 5 months after construction, and involved a length of about 8 ft in which there were four closely spaced transverse cracks. The failure became progressively worse, and in March 1950, the eastbound lane had developed to the condition shown in Fig. 4. Attention is called to the localized nature of the failure and that the pavement in the background is in excellent condition.

The other failure involved two transverse cracks about 3 ft apart in the eastbound lane. Excessive deflection and pumping also occurred at this location, with resulting deep spalling of the transverse cracks.

About May 1, 1950, the condition of the pavement had become so bad at both of these locations that it became necessary to remove the broken concrete and replace it with concrete patches. In neither case was it necessary to patch the westbound lane. Fig. 5 shows the concrete patch placed in the eastbound lane to repair the localized failure shown in Fig. 4.

SUMMARY

The experimental pavement is not old enough to justify a definite conclusion as to its ultimate performance but, based on its present behavior, it would appear that a properly designed continuously reinforced concrete pavement should give excellent performance. In view of the high steel stresses measured in 0.7 percent steel, it may be necessary to provide more steel than was indicated by earlier experiments. It also may be found that continuous reinforcement, while it may be a factor in controlling pumping, cannot by itself economically solve this problem and, therefore, it may be necessary in future experiments to revise the design accordingly. Perhaps over-all performance would be improved by a combination of continuous reinforcement and a thin granular sub-base. These possibilities should be studied in other projects. It is interesting to note in this connection that in 1951 the Texas Highway Department constructed a continuously reinforced pavement 8 in. thick, with 0.7 percent of steel, on 8 in. of granular sub-base on the Fort Worth Urban Expressway.

In the final analysis the criterion by which this type of construction is judged will be an economic one. Whether pavements of this type are practical will depend on their original cost, service life and the cost of maintenance. In other words, will it give more and improved service for the money expended than other types of pavement.

A study of the contract unit prices for the experimental pavement and the adjacent standard pavement, built under the same contract, show that the contractor bid approximately the same unit price for the 7-in. pavement with 0.7 percent steel, the 8-in. pavement with 0.5 percent steel, and the standard 10-in. mesh reinforced pavement with 6-in. granular sub-base. Neither the experimental nor the standard pavement has required much maintenance to date and, of course, no estimate can be made at this time of the probable service life of the various test sections of continuously reinforced pavement. It appears that further study and additional experimental projects will be required before the question of economics can be answered.

Title No. 48-44

Effects of Temperature Changes on Concrete as Influenced by Aggregates*

By STANTON WALKER,† D. L. BLOEM† and W. G. MULLEN‡

SYNOPSIS

Summarizes tests of concrete and mortar exposed to water and air temperatures ranging from 40–140 F with varying rates of change in temperature. Changes in length, weight, dynamic modulus and flexural strength were measured on specimens containing several different fine and coarse aggregates having thermal coefficients of expansion ranging from relatively low to relatively high. Principal findings of the tests were: thermal coefficients of expansion of concrete and mortar containing different aggregates varied approximately in proportion to the thermal coefficient and quantity of aggregate in the mixture; an approximation of the thermal coefficient of expansion of aggregate may be made from determinations of the thermal coefficients of concrete of varying proportions; changes in temperature were destructive to the concrete with sudden changes being much more severe than slower ones; and concretes having higher coefficients of expansion were less resistant to temperature changes than concretes with lower coefficients. No relationship was found between resistance of concrete to temperature changes and differences between thermal coefficients of aggregates and mortar.

INTRODUCTION

Thermal properties of concrete and its ingredients appear to bear some not-well-defined relationships to the durability of concrete. It has been suggested that differences in coefficients of expansion of ingredients set up stresses which contribute to disintegration.[1,2,3] However, such relationships as have been suggested do not appear to be firmly enough supported to warrant application to individual combinations of materials. Effects of thermal properties have been difficult to evaluate because of the influence of other factors.

This paper deals with a limited investigation of concretes and mortars subjected to slow and rapid changes of temperature in air and in water. A range in thermal properties was produced by the use of different aggregates and proportions. The data furnish good information on coefficients of expansion of concrete made with varying proportions of different aggregates. They indicate that quick changes in temperature are destructive and that

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concretes having low coefficients are more resistant to such changes than those having high coefficients. No support is offered for the suggestion that differences in thermal coefficients of ingredients, within the range investigated, are of substantial significance to concrete durability. The investigation shows that an approximation of the thermal coefficient of either fine or coarse aggregate can be made from measurement of length changes with temperature of mortar or concrete.

SCOPE OF TESTS

The studies were in three principal parts:

1. Effects of different temperature-change cycles on changes in length, modulus of elasticity, moisture content and flexural strength of concrete.
2. Effects of quantity of fine aggregate in mortar and of coarse aggregate in concrete on thermal coefficients, for different conditions of saturation.
3. Comparison of resistance to temperature change of concretes made with a variety of aggregates differing as to thermal coefficient.

Aggregates were selected to provide a significant range in thermal coefficients. The cements were blends of four or five brands from the local market. Except where aggregate quantity was a variable, the concretes were proportioned to have a cement content of 5½ sacks per cu yd and a slump of 3 to 4 in. The maximum size of coarse aggregate was 1 in. In the study of fine aggregate quantity, cement paste having a water-cement ratio of 0.4 by weight was used throughout with various quantities of different fine aggregates, the consistency of the mixture being allowed to vary. Where quantity of coarse aggregate was the variable, a mortar of fixed proportions (1:2.85 by absolute volume with water-cement ratio 0.52 by weight) was mixed with various quantities of different coarse aggregates, the consistency, again, being allowed to vary.

Procedures for mixing, testing and curing concrete and mortar were those of the ASTM wherever applicable methods existed. Temperature changes were produced in thermostatically controlled air or water baths. In general, length changes were measured with a Whittemore strain gage between brass inserts 10 in. apart in the 3 x 16-in. faces of 3 x 4 x 16-in. beams. In a few cases, length changes of mortar were measured on 1 x 1 x 11¾-in. bars in a vertical comparator. The modulus of elasticity was determined by dynamic methods in accordance with ASTM Method C 215-47 T. Moduli of rupture of 3 x 4 x 16-in. beams were determined using third-point loading on a 15-in. span with a 4-in. depth.

Details concerning aggregate grading and characteristics of aggregates, concrete and mortar are given in a thesis by W. G. Mullen.[4]

In general three or more specimens were made for each condition. Reproducibility of results was satisfactory and only averages are shown in the tables and diagrams. Length changes were measured with a dial reading to 0.0001 in. over a 10-in. gage length. Therefore, a change of one division on the dial for a temperature change of 100 F represents a unit length change of 0.1×10^{-6} in. per in. per deg F.

STUDIES OF DIFFERENT TEMPERATURE CHANGE CYCLES

The investigation (Series J 41) of effects of different temperature-change cycles was made on four classes of concrete. These were normal and air-entraining concretes made with two aggregate combinations; (1) a nearly pure quartz sand and gravel, and (2) a crushed trap rock with a calcareous sand of glacial origin. These aggregate combinations represented, respectively, materials having relatively high and low thermal coefficients. The fine aggregates were similarly, although not identically, graded. The coarse aggregates were separated into different sizes and recombined to the same grading.

A single batch of each class of concrete was mixed, from which twenty-four 3 x 4 x 16-in. beams were molded. Specimens from each class of concrete, in groups of three, were exposed to different temperature-change cycles as shown in Table 1.

For all cycles except No. 6, specimens were exposed to the temperature indicated for either 7 or 17 hours, to conform with the working day—more than enough time in all cases to attain temperature equilibrium. Except for cycle No. 6, the specimens were cured for 35 days in a standard moist room. In the case of No. 6, only one class of concrete was used, non-air-entraining with quartz sand and gravel, and the curing period in the standard moist room was 96 days.

Temperature-change exposures were continued through 35 cycles or until dynamic moduli of elasticity had been reduced 50 percent. All specimens were tested for flexural strength when temperature-change cycles were discontinued. Control specimens cured under standard conditions were tested for strength and modulus of elasticity at the end of 35 and 96-day curing periods.

TABLE 1—CHARACTERISTICS OF TEMPERATURE-CHANGE EXPOSURES (SERIES J 41)*

	Temperature-change cycle No.					
	1	2	3	4	5‡	6
Cooling cycle						
Medium	Air	Air	Water	Air	Air-water-air	Water
Final temp., deg F	0	0	40	40	40	60
Time, hours	17	17	7	7	31	2
Avg. rate temp. change, deg F per min.†	¼	¼	4	½	½	1
Heating cycle						
Medium	Water	Air	Water	Air	Air-water-air	Water
Final temp., deg F	40	40	140	140	140	160
Time, hours	7	7	17	17	41	2
Avg. rate temp. change, deg F per min.†	1	¼	4	1	1	1

*For all cycles except No. 6, change between cooling and heating phase involved transfer of specimens from one environment to other. For cycle No. 6, specimens remained in water bath, the temperature of which was alternately raised and lowered.

†Values are average time-rate of temperature change at center of specimen. For cycles 1 through 5, actual rates of change were higher at beginning and lower at end of exposure period and were different at different depths below the surface of the specimen. For cycle 6, rate of change was essentially uniform and sufficiently slow so that temperature differences within the specimens were small.

‡Sequence of exposure as follows; air at 40 F, water at 40 F, air at 40 F, air at 140 F, water at 140 F, air at 140 F and repeat.

Table 2 shows the effects of the several exposures all of which, of course, caused changes in length with temperature. Exposure 3, which involved rapid temperature changes over a range of about 100 F, was the only one which caused serious distress in the concrete. Exposures 1 and 2, in which the temperature passed through the freezing point, caused essentially no residual length changes and little reduction in dynamic modulus or flexural strength of the concrete. Exposure 4, which consisted of air temperature changes over a range of about 100 F, caused a slight shrinkage of the concrete and some increase in flexural strength but had no appreciable effect on the dynamic modulus. Exposure 5, which involved wetting and drying as well as heating and cooling, caused no appreciable residual length change but did produce small reductions in dynamic modulus and flexural strength. Exposure 6, which involved gradual temperature changes in water over a range of 100 F, caused no significant expansion but reduced dynamic modulus 8 percent and flexural strength 17 percent in the single type of concrete tested, *i.e.*, normal concrete with quartz aggregates.

Table 3 shows thermal coefficients of expansion and contraction of the four classes of concrete for the six exposures. Thermal coefficients were affected by the type of temperature-change cycle as well as by the class of concrete. Coefficients of normal and air-entraining concretes containing the same aggregates and tested under the same conditions were essentially the

TABLE 2—COMPARISON OF TEMPERATURE-CHANGE CYCLES (SERIES J 41)*

Temperature cycle	Aggregates†	Total growth, in./in. × 10^{-6}	Coeff. expansion, per deg F × 10^{-6}	Coeff. contraction, per deg F × 10^{-6}	Total weight change, g	Dynamic *E*, percent original	Modulus of rupture, percent control
1—40 F water, 0 F air	Quartz	0.2	7.55	7.65	−17	99	88
	Trap + Calc.	0.6	6.44	6.32	−14	95	84
	Avg.	0.4	6.99	6.99	−16	97	86
2— 40 F air, 0 F air	Quartz	0.6	7.30	7.26	−33	101	104
	Trap + Calc.	0.3	5.38	5.52	−40	100	97
	Avg.	0.5	6.34	6.39	−36	100	100
3—140 F water, 40 F water	Quartz‡	5.4	6.44	5.56	+16	53	29
	Trap + Calc.	6.2	4.51	4.12	+ 7	88	40
	Avg.	5.8	5.48	4.84	+11	70	34
4—140 F air, 40 F air	Quartz	−0.3	6.03	5.92	−60	98	109
	Trap + Calc.	−0.3	4.64	4.78	−76	102	131
	Avg.	−0.3	5.38	5.35	−68	100	120
5— 40 F water-air, 140 F air-water and back	Quartz	0.2	6.18	6.14	−39	92	86
	Trap + Calc.	0.6	4.52	4.46	−38	98	95
	Avg.	0.4	5.35	5.30	−38	95	90
6—160 F to 60 F water	Quartz§	0.6	6.60	6.63	−13	92	83

*Specimens 3 x 4 x 16-in. concrete beams. All specimens cured 35 days in standard moist room before being subjected to exposure, except that concrete for exposure 6 was cured 96 days. All values average of 6 beams, 3 air-entraining and 3 non-air-entraining. Specimens subjected to 35 cycles except as noted.

†Quartz sand and gravel or trap rock coarse aggregate plus calcareous sand.

‡Fifteen cycles only. Removed when Dynamic *E* reached about 50 percent.

§Single type of concrete tested in cycle 6.

same. Except in Exposure 3, where considerable residual expansions developed, coefficients of expansion and contraction were essentially the same for given conditions. Accordingly, attention is directed to the value in Table 3 which represents the average of normal and air-entraining concrete and of both expansion and contraction.

The coefficients of expansion were significantly higher when determined within the range of 0 and 40 F than for the range from about 40 to 140 F. This may be in part because in the first instance the concrete was frozen and thawed, and in part because of the different ranges in temperature—about 40 F as compared with 100 F. The two freezing and thawing cycles gave about the same values of thermal coefficient for the quartz aggregate combination. In the trap rock combination, for reasons not apparent, the concrete thawed in water gave a higher coefficient than that thawed in air.

The three cycles above freezing and of a range of approximately 100 F (No. 3, 4 and 5) all gave about the same thermal coefficients. Exposure 6, which involved a slow change of temperature in water and which was made for the normal concrete with the quartz aggregate only, showed an intermediate thermal coefficient.

As already pointed out, Exposure 3 which involved a rapid change in temperature over the range of about 100 F resulted in significant damage to the concrete. Table 4 and Fig. 1 and 2 summarize the results.

It can be seen in Fig. 1 that the moduli of elasticity of the concretes made with the quartz aggregates were reduced quite rapidly—50 percent in about 15 cycles. The concretes made with the trap rock and calcareous sand combination showed considerably better resistance to this rapid temperature

TABLE 3—COMPARISON OF THERMAL COEFFICIENTS OF CONCRETE (SERIES J 41)*

Temperature cycle	Direction of length change	Coefficient of length change per deg F × 10^{-6}					
		Quartz sand and gravel			Trap rock + calc. sand		
		Normal conc.	A-E conc.	Avg.	Normal conc.	A-E conc.	Avg.
1— 40 F water, 0 F air	Expansion	7.41	7.69	7.55	5.99	6.88	6.44
	Contraction	7.39	7.92	7.65	5.75	6.90	6.32
	Avg.	7.40	7.80	7.60	5.87	6.89	6.38
2— 40 F air, 0 F air	Expansion	7.16	7.45	7.30	5.25	5.51	5.38
	Contraction	7.12	7.39	7.26	5.59	5.44	5.52
	Avg.	7.14	7.42	7.28	5.42	5.48	5.45
3—140 F water, 40 F water	Expansion	6.49	6.40	6.44	4.54	4.48	4.51
	Contraction	5.59	5.53	5.56	4.13	4.10	4.12
	Avg.	6.04	5.96	6.00	4.34	4.29	4.32
4—140 F air, 40 F air	Expansion	5.97	6.09	6.03	4.56	4.71	4.64
	Contraction	5.90	5.93	5.92	4.75	4.82	4.78
	Avg.	5.94	6.01	5.98	4.66	4.76	4.71
5— 40 F water to 40 F air to 140 F air to 140 F water and back	Expansion	6.18	6.19	6.18	4.50	4.53	4.52
	Contraction	6.04	6.24	6.14	4.46	4.47	4.46
	Avg.	6.11	6.21	6.16	4.48	4.50	4.49
6—160 F to 60 F in water	Expansion	6.60†					
	Contraction	6.63					
	Avg.	6.62					

*See Table 2 and text for details of tests.
†Single type of concrete tested in cycle 6.

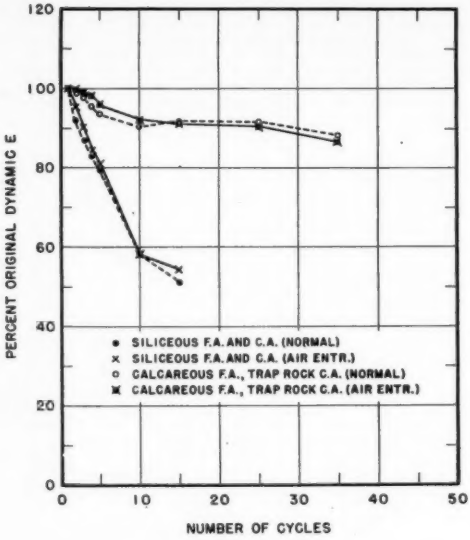
PERCENT ORIGINAL DYNAMIC E
NUMBER OF CYCLES
SILICEOUS F.A. AND C.A. (NORMAL)
SILICEOUS F.A. AND C.A. (AIR ENTR.)
CALCAREOUS F.A., TRAP ROCK C.A. (NORMAL)
CALCAREOUS F.A., TRAP ROCK C.A. (AIR ENTR.)

Fig. 1—Effect of exposure 3 on modulus of elasticity of concrete (series J 41)

change, their dynamic moduli having been reduced only 12 percent after 35 cycles, when the tests were discontinued. Air entrainment appeared to have no significant effect on the results. All of the concretes developed fine map cracks after only 2 cycles and these increased in amount and size throughout the remainder of the test. From Fig. 2 it will be observed that this temperature-change cycle resulted in residual expansions and that these were greater for the quartz aggregate combination than for the trap-rock calcareous-sand combination.

These data and those in Table 2 suggest that it is the rapid changes in temperature which are principally destructive. Table 2 shows no significant differences between the two aggregate combinations when temperature

TABLE 4—EFFECT OF RAPID HEATING AND COOLING ON LENGTH CHANGE AND MODULUS OF ELASTICITY OF CONCRETE (SERIES J 41)*

	Cumulative growth, in./in. × 10^{-4}				Dynamic modulus of elasticity, percent			
	Quartz sand and gravel		Trap rock + calc. sand		Quartz sand and gravel		Trap rock + calc. sand	
Cycles	Normal conc.	A-E conc.	Normal conc.	A-E conc.	Normal conc.	A-E conc.	Normal conc.	A-E conc.
0	—	—	—	—	100	100	100	100
1	0.0	0.0	0.0	0.0	100	100	100	100
2	0.7	1.1	0.6	0.4	92	95	99	100
3	1.6	1.6	1.1	0.8	87	90	98	99
4	2.3	2.4	1.6	1.1	83	85	96	98
5	2.9	3.2	2.0	1.4	80	80	93	96
10	4.9	4.3	2.8	2.6	58	58	90	92
15	5.7	5.2	3.5	3.3	51	55	92	92
25			5.8	5.0			92	91
35			6.6	5.8			88	87

*Specimens 3 x 4 x 16-in. beams, cured 35 days in standard moist room. Specimens alternately placed in water at about 140 F for 17 hours and in water at about 40 F for 7 hours. Each value average for 3 beams.

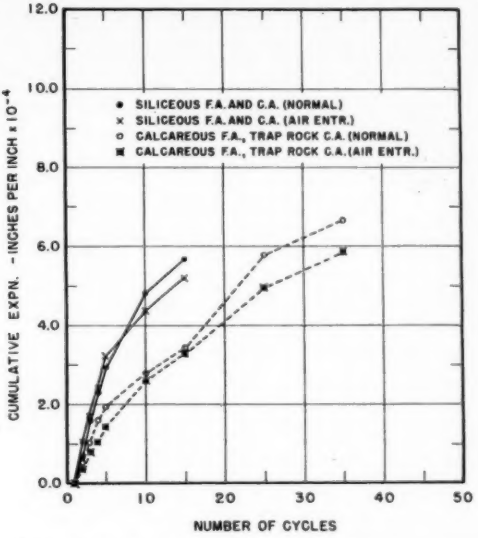

Fig. 2—Effect of exposure 3 on concrete expansion (series J 41)

changes were relatively slow (Cycles 1, 2, 4, 5 and 6). Table 4 and Fig. 1 and 2, on the other hand, show a very marked difference in the action of the two aggregate combinations as to residual expansion and reduction in modulus of elasticity for the rapid changes in temperature produced in Cycle 3.

The coefficients of expansion in Table 3 appear to offer the best available explanation for these differences in resistance to the "shock" treatment given in Cycle 3. The thermal coefficient of the quartz aggregate concrete was about 7.4×10^{-6} and that of the trap rock, calcareous sand concrete was about 5.9×10^{-6} when determined in the 0 to 40 F range and 6.0×10^{-6} and 4.5×10^{-6}, respectively, when determined in the higher range. That is to say, the quartz aggregate concrete had a thermal coefficient of expansion approximately 30 percent higher than that of the trap-rock calcareous-aggregate combination.

This investigation offers little information on the effects of differences in thermal coefficient between coarse aggregate and mortar on resistance of the concrete to temperature changes. For both aggregate combinations, the coefficients of expansion of coarse aggregate, mortar and concrete were approximately the same. In the case of the quartz sand and gravel combination, the thermal coefficients in millionths of an inch per inch per deg F were about 6.3 for the coarse aggregate, 6.1 for the mortar and 6.0 for the concrete. Corresponding values for the trap rock, calcareous sand combination were 3.9, 4.5 and 4.5. In this case at least, differences in resistance to temperature changes appear to have been caused by differences in thermal coefficients of the concretes themselves rather than to lack of similarity of the thermal coefficients of ingredients.

COEFFICIENTS OF EXPANSION AS AFFECTED BY PROPORTIONS

It has been well demonstrated by many investigators that length, or volume, changes in concrete due to temperature are closely related to the volume change characteristics of the ingredients. It seems reasonable to expect that the thermal coefficients of length change of concrete or mortar will approximate the average of the thermal coefficients of the ingredients, weighted in accordance with their absolute volumes. If that is so, then order-of-magnitude determinations of the thermal coefficients of fine and coarse aggregates may be made from measurements on mortar or concrete of different proportions.

This relationship was investigated. The mortars were made with a natural quartz sand, a crushed limestone sand and a crushed trap rock sand. Concretes were made with quartz gravel and crushed limestone, each being added to a quartz sand mortar and to a mortar made with the crushed limestone sand. In general, three rounds of specimens were made. The quartz materials were representative of fairly high thermal coefficients and the limestone and trap rock of fairly low coefficients.

In the mortar tests, the fine aggregates were added to a cement paste of 0.4 water-cement ratio by weight in amounts of 0, 20, 40 and 60 percent of the absolute volume of the resulting mortar. In the concrete tests, the mortars were proportioned 1 part of cement to 2.85 parts of fine aggregate by absolute volume with a water-cement ratio of 0.52 by weight. Each of the two coarse aggregates was used with the two mortars in quantities of 0, 20, 40 and 50 percent of the volume of the resulting concrete.

Specimens for the fine aggregate study were 1 x 1 x 11¾-in. bars. Specimens for the coarse aggregate study were 3 x 4 x 16-in. beams. Specimens were moist cured at 70 F (the work schedule resulted in 44 days for the mortar specimens and 70 days for the concrete) and then subjected to 10 cycles of slow heating and cooling in water over the range of about 60 to 140 F. The rate of temperature change was about ¼ deg F per minute. Average thermal coefficients were calculated from length measurements made at the maximum and minimum temperatures for each of the 10 cycles.

The specimens developed no significant residual expansions, reductions in modulus of elasticity or other evidences of distress due to the treatment just described. After that treatment, the concrete specimens were air-dried at 70 F for 57 days and then subjected to slow heating and cooling in air over about the same range of temperature (in this case from about 45 to 135 F) for 10 cycles, with measurements being taken as before. They were then dried for an additional 195 days (15 at 70 F, 113 at 120 F, and 67 at 100 F) and again subjected to 10 cycles of slow heating and cooling in air. Since temperatures in the air chambers were controlled by means of a surrounding water jacket, relative humidity was necessarily high. Excellent agreement was found between measurements for companion specimens and during successive cycles for the same treatment.

TABLE 5—THERMAL COEFFICIENTS OF LINEAR EXPANSION OF SATURATED MORTAR BARS (SERIES 127A)*

Fine aggregate, percent volume of mortar	Coefficient of expansion per deg F × 10^{-6}		
	Quartz sand, Lot 2124	Mfd. limestone sand, Lot 2241	Mfd. trap rock sand, Lot 2273
0†	5.40	5.40	5.40
20	5.57	4.67	4.73
40	5.52	3.81	4.04
60	5.79	3.33	3.73
100‡	6.30	2.45	3.91

*Specimens 1 x 1 x 11¾-in. bars except as noted. All specimens made with W/C of 0.40 by weight. Specimens cured 44 days in standard moist room before ten daily cycles of heating and cooling in water. Each value is an average of 20 determinations on each of 3 specimens from separate batches mixed on different days.

†Neat cement bars, no fine aggregate.

‡Coefficient for quartz sand represents values given in standard handbooks. Coefficients for limestone and trap rock determined during slow heating and cooling in water of prisms cut from the parent rock along 3 mutually perpendicular axes.

The results of the mortar tests are shown in Table 5 and Fig. 3. Those for the concrete tests are shown in Table 6 and in part, in Fig. 4. The value of thermal coefficient of expansion shown for quartz, both fine and coarse aggregate, is one common in standard handbooks. Those given for the limestone and trap rock, both fine and coarse, were determined by direct

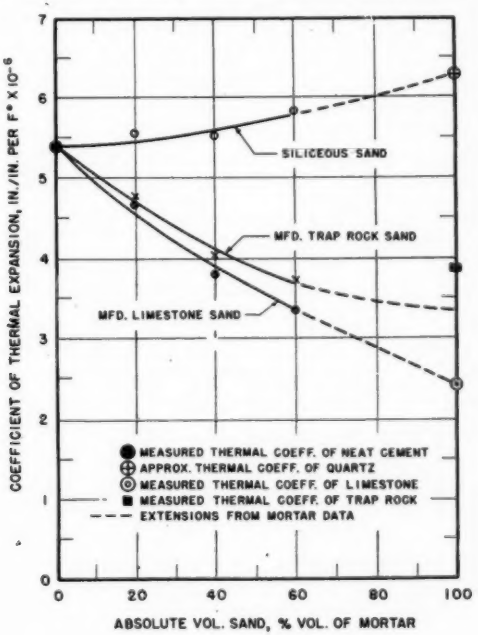

Fig. 3—Relationship between thermal coefficient of mortar and quantity and type of sand (series 127, saturated mortar)

TABLE 6—THERMAL COEFFICIENTS OF LINEAR EXPANSION FOR CONCRETES IN DIFFERENT MOISTURE CONDITIONS (SERIES 127)*

Coarse aggregate	Fine aggregate	Moisture condition of concrete	Coarse aggregate, percent volume of concrete				
			0†	20	40	50	100‡
			Coefficient of expansion per deg F × 10^{-6}				
Quartz gravel, Lot 2147	Quartz sand, Lot 2124	Saturated	6.14	6.14	6.24	6.22	6.3
		Partially dry	8.53	7.81	7.30	7.24	
		Oven dry	6.79	6.59	6.35	6.32	
Crushed limestone, Lot 2241	Quartz sand, Lot 2124	Saturated	6.14	5.13	4.33	4.02	2.45
		Partially dry	8.53	6.20	4.53	4.28	
		Oven dry	6.79	5.23	4.12	3.71	
Quartz gravel, Lot 2147	Mfd. limestone, Lot 2241	Saturated	3.98	4.47	4.84	5.08	6.3
		Partially dry	5.35	5.61	5.76	5.81	
		Oven dry	4.38	5.06	5.40	5.60	
Crushed limestone, Lot 2241	Mfd. limestone, Lot 2241	Saturated	3.98	3.78	3.58	3.42	2.45
		Partially dry	5.35	4.57	4.04	3.67	
		Oven dry	4.38	4.00	3.62	3.51	

*Specimens 3 x 4 x 16 in. beams except as noted. Specimens made with mortar proportioned 1:2.85 by absolute volume with W/C of 0.525 by weight. Each value average of at least 10 determinations on each of 3 specimens during slow heating and cooling cycles.

†Mortar beams, no coarse aggregate.

‡Coefficient for quartz gravel represents values given in standard handbooks. Coefficient for limestone determined during slow heating and cooling in water or air of prisms cut from the parent rock along 3 mutually perpendicular axes.

measurement of specimens sawed from the parent stone. They represent the average of measurements taken along three mutually perpendicular axes.

It is apparent from Fig. 3 and 4 that aggregates affect the thermal coefficients of mortar and concrete approximately in proportion to their quantity in the mixture. The extension of these curves to the abscissa representing 100 percent aggregate gives values for thermal coefficient which agree reasonably well with those determined by direct measurement on the rocks in question. This supports the suggestion that the thermal coefficient of concrete is approximately the weighted average of the coefficients of its ingredients. Further, it supports the thought that measurements on concrete and mortar can be used to make order-of-magnitude determinations of the thermal coefficients of aggregates.

A comparison of the values for mortar given in Fig. 3 and 4 show that there are inconsistencies, probably resulting from the differences in specimens used—1 x 1-in. bars in one case and 3 x 4-in. beams in the other. Also, it should be noted that the smaller specimens were moist-cured for 90 days as against only 70 days for the larger ones. The data plotted in Fig. 4 are for the saturated concrete since those values are most readily compared with data for the mortar. However, as pointed out and as shown in Table 6, measurements were made for different moisture conditions. Thermal coefficients determined for concretes in a saturated condition and after thorough drying were in reasonably good agreement. Those determined for the partially dry condition were considerably higher, a phenomenon noted by other investigators.[5]

In spite of some very large differences in thermal coefficient between mortar and coarse aggregate, none of the concretes developed evidence of deterioration during the 30 cycles of heating and cooling. As shown in Table

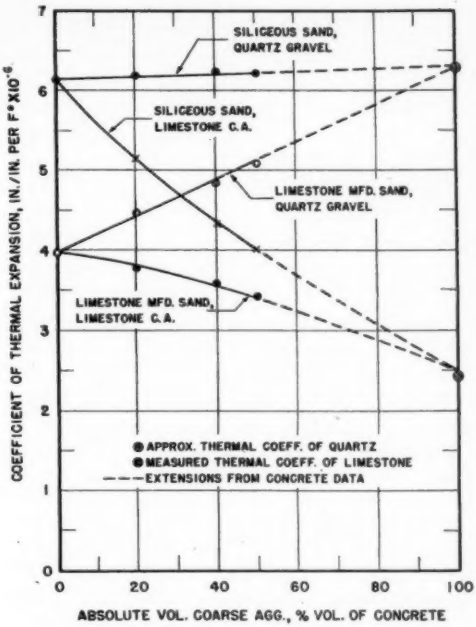

Fig. 4—Relationship between thermal coefficient of concrete and quantity and type of aggregate (series 127, saturated concrete)

6, thermal coefficient differences in excess of 6×10^{-6} per deg F were found for partially dry concretes and as high as 3.7×10^{-6} per deg F for saturated concretes. It appears that, for the relatively slow rate of heating and cooling employed in this phase of the investigation, the thermal coefficient of the concrete or its ingredients was not important.

TEMPERATURE CHANGE EFFECTS ON CONCRETE CONTAINING DIFFERENT AGGREGATES

Two groups of tests (Series 135 and 135B) were made to study the influence of various aggregate combinations on the resistance of concrete to both a fast and a moderately slow cycle of heating and cooling.

In Series 135, combinations of 3 coarse and 2 fine aggregates were studied. The coarse aggregates were quartz gravel, crushed limestone and crushed trap rock and the fine aggregates were quartz sand and manufactured limestone sand—all used in earlier phases of the investigation.

Three batches of concrete were mixed on different days for each aggregate combination. Two 3 x 4 x 16-in. beams were molded from each batch and cured to the age of 28 days in the standard moist room at 70 F. One beam was then subjected to rapid heating and cooling in water through the range of about 40 to 140 F, specimens being transferred between water baths maintained at the two temperature extremes (Cycle 3, described earlier).

The second beam was exposed to heating and cooling at a slower rate through the same temperature range by gradual changes in temperature of a water bath in which the specimens were kept continuously immersed (Cycle 6A). This cycle was similar to No. 6, described earlier, with the temperature (over a slightly different range) being changed from about 40 to 140 F at the rate of approximately 1 deg F per minute. This has been referred to as a slow cycle which it is, relative to Cycle 3. However, it is much more rapid than would ordinarily be encountered in nature.

Effects of the two exposures on the concretes are summarized in Table 7. As had been found previously, the rapid temperature changes of cycle No. 3 were highly destructive regardless of which aggregates were used. The dynamic moduli of all concretes were greatly reduced and all developed excessive expansions during their exposure, which was for 98 cycles or until dynamic modulus had been reduced about 50 percent. All specimens showed a reduction of 70 to 80 percent in modulus of rupture at that cycle when heating and cooling was discontinued.

Fig. 5 and 6 show relationships between the thermal coefficients of the concrete and its ingredients and the resistance of the concrete to rapid tem-

TABLE 7—COMPARISON OF RESISTANCE TO TEMPERATURE CHANGES OF CONCRETES CONTAINING DIFFERENT AGGREGATE COMBINATIONS (SERIES 135)*

Fine aggregate	Coarse aggregate	Coefficient of expansion per deg F × 10^{-6}				No. of cycles§	Expansion, in./in. × 10^{-4}**	Dynamic, E, percent original**	Modulus of rupture, percent original**
		Coarse aggregate†	Mortar‡	Concrete‡	C. A.-Mortar				
Cycle 3, 40 F to 140 F water (rapid)									
Quartz sand	Quartz	6.3	5.8	5.8	+0.5	48	8.9	48	28
	Limestone	2.4	5.8	3.9	−3.4	86	9.9	53	27
	Trap rock	3.9	5.8	4.6	−1.9	65	8.6	47	30
	Avg.	4.2	5.8	4.8	−1.6	66	9.1	49	28
Mfd. limestone	Quartz	6.3	3.6	4.4	+2.7	87	16.2	53	20
	Limestone	2.4	3.6	3.2	−1.2	98	9.8	72	26
	Trap rock	3.9	3.6	3.5	+0.3	98	13.8	64	29
	Avg.	4.2	3.6	3.7	+0.6	94	13.3	63	25
Avg.	Quartz	6.3	4.7	5.1	+1.6	68	12.6	50	24
	Limestone	2.4	4.7	3.6	−2.3	92	9.8	62	26
	Trap rock	3.9	4.7	4.0	−0.8	82	11.2	56	30
Cycle 6A, 40 F to 140 F in water (slow)									
Quartz sand	Quartz	6.3	5.8	5.8	+0.5	98	2.0	94	72
	Limestone	2.4	5.8	3.9	−3.4	98	2.1	94	76
	Trap rock	3.9	5.8	4.6	−1.9	98	2.0	89	68
	Avg.	4.2	5.8	4.8	−1.6	98	2.0	92	72
Mfd. limestone	Quartz	6.3	3.6	4.4	+2.7	98	3.1	102	75
	Limestone	2.4	3.6	3.2	−1.2	98	1.5	103	100
	Trap rock	3.9	3.6	3.5	+0.3	98	1.5	103	101
	Avg.	4.2	3.6	3.7	+0.6	98	2.0	103	92
Avg.	Quartz	6.3	4.7	5.1	+1.6	98	2.6	98	74
	Limestone	2.4	4.7	3.6	−2.3	97	1.8	98	88
	Trap rock	3.9	4.7	4.0	−0.8	98	1.8	96	84

*Each value average for three 3 x 4 x 16-in. beams from separate batches of concrete mixed on different days.
†See Table 5 for source of data.
‡Thermal coefficients of mortar and concrete determined in cycle 6A.
§Exposure continued through 98 cycles or until dynamic modulus had been reduced about 50 percent.
**After number of cycles indicated.

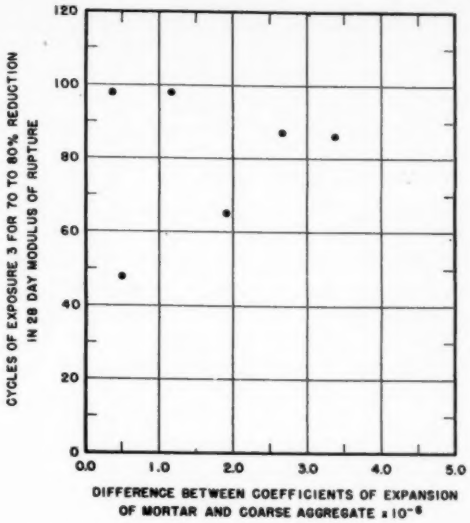

Fig. 5—Effect of coefficient difference on resistance of concrete to exposure 3 (series 135)

perature changes. It can be seen in Fig. 5 that there is no correlation between the rate at which the concrete deteriorated and differences between thermal coefficients of coarse aggregate and mortar. Fig. 6, however, reveals an excellent correlation between the thermal coefficient of the concrete itself and

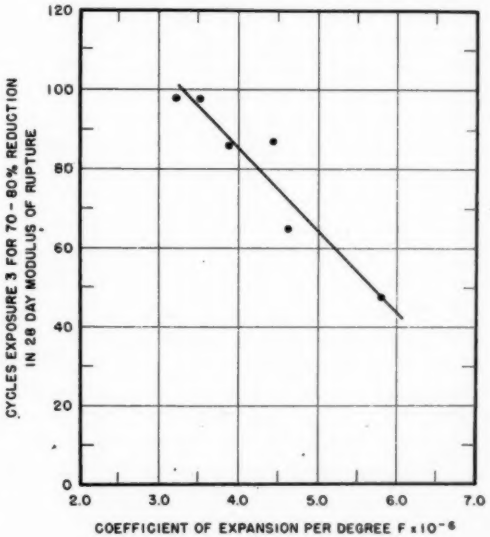

Fig. 6—Effect of coefficient of expansion on resistance of concrete to exposure 3 (series 135)

its resistance to the rapid temperature changes, resistance decreasing as thermal coefficient increased. The data suggest that rapid disruption of the concrete was caused by the large differential strains developed when the surrounding temperature was changed quickly.

Table 7 shows that resistance of all concrete to slow temperature changes (Cycle 6A) was very much better than for the rapid changes (Cycle 3). There were no excessive residual expansions or reductions in dynamic modulus. There were, however, reductions of about 25 to 30 percent in modulus of rupture for all concretes except the two which had coefficients of expansion appreciably less than 4×10^{-6} per deg F. It is interesting to note that such evidence of disintegration as there was manifested itself only in the four concretes having the highest coefficients of expansion and that there is no apparent correlation between disintegration and thermal coefficient differences between coarse aggregate and mortar. This is in the direction of supporting the trends noted for Cycle No. 3.

Series 135B was similar to Series 135 with respect to testing procedures, but additional aggregates were included to give a more complete range of thermal coefficients of coarse aggregate and mortar. All combinations of 4 fine and 4 coarse aggregates were investigated. The fine aggregates were natural quartz and calcareous sands (used previously) and manufactured granite and limestone sands (not previously used). In addition to the quartz gravel used earlier, chert gravel, crushed limestone and crushed granite coarse aggregates from new sources were included. The fine limestone and granite aggregates were from the same parent rock as their respective coarse aggregates. Apparently the new limestone had a higher thermal coefficient than the one used in the earlier tests with the result that no very large differences between thermal coefficients of coarse aggregate and mortar were produced. Nevertheless the data appear of sufficient interest to bear examination.

Table 8 shows the effects of rapid temperature changes between 40 and 140 F in water (Cycle 3). As was the case in the earlier tests, this cycle was highly destructive. All concrete, with thermal coefficients ranging from about 4 to 6×10^{-6} per deg F, had developed excessive permanent expansion and reduction in modulus of rupture when the tests were discontinued at 50 cycles or earlier. In general the concretes with the higher coefficients showed the greater reduction in modulus of rupture, but that was not true of residual expansion, where the limestone aggregate concrete increased most in length.

There appears to be no very large nor consistent difference in the resistance to rapid temperature changes attributable to the various aggregates *per se*. On the basis of expansion and modulus of rupture, the calcareous sand and manufactured granite fine aggregates gave about the same average results and were somewhat better than the manufactured limestone and quartz sands; on the basis of residual dynamic modulus, the calcareous sand was highest, manufactured limestone and granite about the same and quartz sand lowest. Among the coarse aggregates there were no large differences among average expansions, dynamic moduli or moduli of rupture but the granite in general

TABLE 8—RESISTANCE TO RAPID HEATING AND COOLING OF CONCRETES CONTAINING DIFFERENT AGGREGATE COMBINATIONS (SERIES 135B)*

Fine aggregate	Coarse aggregate	Coefficient of expansion per deg F × 10-6†				No. of cycles‡	Expansion, in./in. × 10-4§	Dynamic, E, percent original§	Modulus of rupture, percent original§
		Coarse aggregate	Mortar	Concrete	C.A.-mortar				
Quartz sand	Quartz	6.3	5.7	5.9	+0.6	45	8.8	58	32
	Chert	6.4	5.7	6.0	+0.7	27	7.6	51	33
	Limestone	4.0	5.7	4.9	−1.7	47	15.0	51	36
	Granite	4.2	5.7	5.2	−1.5	50	7.9	62	38
	Avg.	5.2	5.7	5.5	−0.5	42	9.8	56	35
Mfd. granite	Quartz	6.3	4.2	4.9	+2.1	50	7.4	62	37
	Chert	6.4	4.2	4.9	+2.2	50	6.1	62	44
	Limestone	4.0	4.2	4.4	−0.2	50	12.5	62	51
	Granite	4.2	4.2	4.1	0.0	50	5.4	70	62
	Avg.	5.2	4.2	4.6	+1.0	50	7.9	64	48
Mfd. limestone	Quartz	6.3	4.3	4.7	+2.0	50	15.0	63	35
	Chert	6.4	4.3	4.9	+2.1	50	13.3	63	36
	Limestone	4.0	4.3	4.3	−0.3	50	14.4	68	33
	Granite	4.2	4.3	4.4	−0.1	50	11.0	73	39
	Avg.	5.2	4.3	4.6	+0.9	50	13.4	67	36
Calcareous sand	Quartz	6.3	4.5	5.0	+1.8	50	9.8	86	41
	Chert	6.4	4.5	5.1	+1.9	50	8.4	78	45
	Limestone	4.0	4.5	4.2	−0.5	50	11.0	85	45
	Granite	4.2	4.5	4.4	−0.3	50	5.5	92	54
	Avg.	5.2	4.5	4.7	+0.7	50	8.7	85	46
Avg.	Quartz	6.3	4.7	5.1	+1.6	49	10.3	67	36
	Chert	6.4	4.7	5.2	+1.7	44	8.8	64	40
	Limestone	4.0	4.7	4.4	−0.7	49	13.2	66	41
	Granite	4.2	4.7	4.5	−0.5	50	7.4	74	48

*Specimens 3 x 4 x 16-in. beams subjected to rapid heating and cooling in water between 140 and 40 F (Cycle 3). Each value average of 3 specimens from separate batches of concrete mixed on different days.
†Values for quartz and chert are from reference sources; those for limestone and granite determined graphically by method demonstrated in Fig. 4; values for mortar and concrete are from measurements made on companion specimens in Exposure 6A (See Table 9).
‡Exposure continued through 50 cycles or until dynamic modulus had been reduced about 50 percent.
§After number of cycles indicated.

gave slightly better performance than the quartz gravel, chert and limestone, all of which were about the same.

Relationships of thermal coefficient characteristics of the concrete and its ingredients to resistance of the concrete to rapid temperature changes are shown in Fig. 7 and 8. These are based on moduli of rupture of the concretes after exposure, expressed as percentages of values for companion specimens tested after 28 days of moist curing. Fig. 7 reveals no distinguishable relationship between resistance to rapid temperature changes and difference between thermal coefficients of coarse aggregate and mortar. Fig. 8 shows a trend indicating that resistance becomes less as the thermal coefficient of the concrete increases. The trend, however, is not so well-defined as the similar relationship found in Series 135, discussed earlier.

Data for the moderately slow heating and cooling cycle (No. 6A) are summarized in Table 9. In this phase of the investigation, as in earlier phases, the slower temperature changes were much less destructive than the rapid ones. Except for concretes containing the crushed limestone coarse aggregates, expansions in no case exceeded 0.05 percent during the 50 cycles of exposure. Reductions in dynamic modulus ranged from 2 to 26 percent and, in modulus of rupture, from 1 to 38 percent.

On the average, concretes made with calcareous sand and manufactured limestone fine aggregates showed slightly less reduction in dynamic modulus

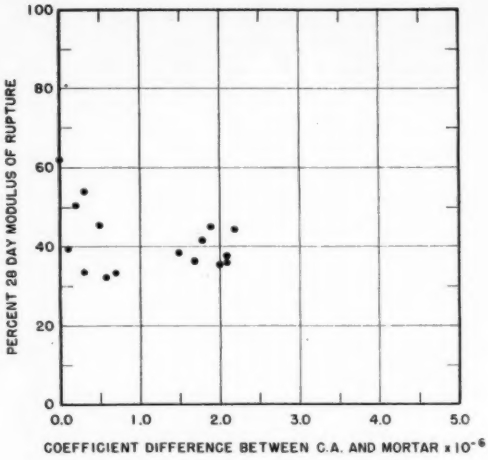

Fig. 7—Relation between reduction in modulus of rupture and coefficient difference after 50 cycles of exposure 3 (series 135B)

of rupture than those with granite or quartz sand. Average results for the coarse aggregates were all about the same except that the quartz gravel showed somewhat greater reduction in modulus of rupture and the crushed limestone greater expansion than the others.

Plotting of the data from Table 9, as was done for the rapid cycle in Fig. 7 and 8, again failed to reveal any relationship between resistance to temperature change and difference between coefficients of expansion of mortar and coarse aggregate. There was only a faint trend suggesting that resistance to the slower heating and cooling cycle was reduced as thermal coefficient of

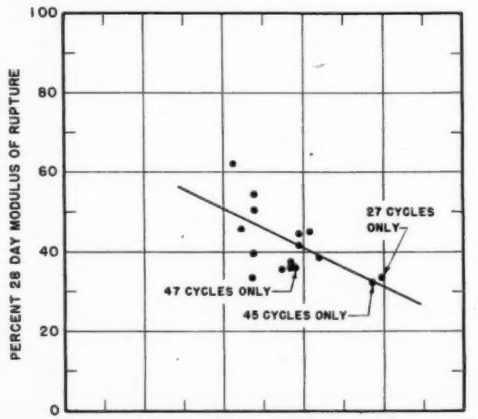

Fig. 8—Relation between modulus of rupture and thermal coefficient of expansion of concrete after 50 cycles of exposure 3 (series 135B)

TABLE 9—RESISTANCE TO SLOW HEATING AND COOLING OF CONCRETES CONTAINING DIFFERENT AGGREGATE COMBINATIONS (SERIES 135B)*

Fine aggregate	Coarse aggregate	Coefficient of expansion per deg F × 10-6†				No. of cycles‡	Expansion, in./in. × 10-4§	Dynamic, E, percent original§	Modulus of rupture, percent original§
		Coarse aggregate	Mortar	Concrete	C.A.-mortar				
Quartz sand	Quartz	6.3	5.7	5.9	+0.6	50	1.6	86	62
	Chert	6.4	5.7	6.0	+0.7	50	1.5	88	77
	Limestone	4.0	5.7	4.9	−1.7	50	8.1	74	65
	Granite	4.2	5.7	5.2	−1.5	50	1.9	83	67
	Avg.	5.2	5.7	5.5	−0.5	50	3.3	83	68
Mfd. granite	Quartz	6.3	4.2	4.9	+2.1	50	2.6	81	72
	Chert	6.4	4.2	4.9	+2.2	50	2.3	81	71
	Limestone	4.0	4.2	4.4	−0.2	50	7.5	74	74
	Granite	4.2	4.2	4.1	0.0	50	1.6	85	79
	Avg.	5.2	4.2	4.6	+1.0	50	3.5	80	74
Mfd. limestone	Quartz	6.3	4.3	4.7	+2.0	50	3.9	90	63
	Chert	6.4	4.3	4.9	+2.1	50	3.3	90	80
	Limestone	4.0	4.3	4.3	−0.3	50	5.5	92	97
	Granite	4.2	4.3	4.4	−0.1	50	3.1	93	95
	Avg.	5.2	4.3	4.6	+0.9	50	4.0	91	84
Calcareous sand	Quartz	6.3	4.5	5.0	+1.8	50	3.9	98	65
	Chert	6.4	4.5	5.1	+1.9	50	1.9	97	95
	Limestone	4.0	4.5	4.2	−0.5	50	5.3	93	99
	Granite	4.2	4.5	4.4	−0.3	50	2.2	95	98
	Avg.	5.2	4.5	4.7	+0.7	50	3.3	96	89
Avg.	Quartz	6.3	4.7	5.1	+1.6	50	3.0	89	66
	Chert	6.4	4.7	5.2	+1.7	50	2.2	89	81
	Limestone	4.0	4.7	4.4	−0.7	50	6.6	83	84
	Granite	4.2	4.7	4.5	−0.5	50	2.2	89	85

*Specimens 3 x 4 x 16-in. beams subjected to slow heating and cooling in water between 140 and 40 F; temperature change rate at center of specimen about 1 F deg per min. (Cycle 6A). Each value average of 3 specimens from separate batches of concrete mixed on different ways.

†Values for quartz and chert are from reference sources; those for limestone and granite determined graphically by method demonstrated in Fig. 4.

‡All specimens subjected to 50 cycles, then tested for flexural strength.

§At end of 50 cycles.

the concrete increased. In the interest of conserving space, these diagrams have been omitted here, but the data for them are available in Table 9 and they are reproduced in the thesis mentioned earlier.[4]

The data from Series 135 and 135B lend support to the indications of earlier phases of the investigation. They suggest that laboratory cycles which produce rapid changes in temperature cause differential strains in concrete specimens which result in disruption and failure quite unrelated to potential field performance. The disintegration of concrete during rapid heating and cooling, and such tendencies as there were toward deterioration during slower heating and cooling, appeared more closely related to the coefficient of expansion of the concrete itself than to differences between coefficients of ingredients.

CLOSURE

The investigation described was not extensive enough, nor were developed relationships clearly enough defined, to provide conclusive evidence regarding the importance of thermal coefficient characteristics to the durability of concrete. General indications, however, suggested that simple relationships among the thermal coefficients of concrete ingredients are not a reliable index of potential resistance to temperature variations. These principal findings may be summarized as follows:

1. A fast rate of temperature change over a given range is much more destructive to concrete than a slow rate. This suggests that differential volume changes among the ingredients are not the primary cause of disintegration.

2. The resistance of concrete to temperature changes appears to be more closely related to the thermal coefficient of the concrete itself than to differences between the thermal coefficients of ingredients. The trend was for concretes of high thermal coefficient to fail more rapidly than those of low thermal coefficient. The relationship was not well enough defined to be used as a quantitative indication of potential resistance to temperature changes.

3. Deterioration of concrete during temperature changes appears to result from strains set up due to differences in temperature within the mass rather than due to differences in thermal coefficients. The rapidity of temperature changes necessary to produce disintegration of concrete in these tests was much greater than would generally be expected in natural exposures, suggesting that coefficient of expansion of concrete is not an important consideration in evaluating potential durability.

4. Coarse aggregates having thermal coefficients different from the mortar with which they were mixed caused changes in the thermal coefficient of the concrete roughly in proportion to the volume of coarse aggregate used. Similarly, fine aggregates affected the thermal coefficient of mortar approximately in proportion to their volume in the mixture. These relationships appeared sufficiently reliable to permit the approximate determination of aggregate coefficients by measurements on concretes and mortars of varying proportions.

5. Thermal coefficients of mortar and concretes were found to be greatly affected by condition of saturation. Coefficients of expansion for the saturated and oven-dry conditions were similar while those for a partially dry condition were considerably higher.

ACKNOWLEDGMENT

The research studies were conducted by W. G. Mullen in partial fulfillment of the requirements for the degree of Master of Science at the University of Maryland. Funds were provided by the Stephen Stepanian Fellowship of the National Ready Mixed Concrete Assn. The tests were made in the laboratory sponsored jointly by that association and the National Sand and Gravel Assn. at the University of Maryland. The work was done under the general direction of the senior author.

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Title No. 48-45

Bond Properties of Welded Wire Fabric*

By ARTHUR R. ANDERSON†

SYNOPSIS

A wealth of information has been published on bond between concrete and bar reinforcement, but little is known about the bond properties of welded wire fabric reinforcement. Welded wire fabric is widely used as concrete reinforcement, but bond properties have not been clearly established or specified in codes.

The tests reported indicated that the bond stress theory as applied to smooth or deformed bars is not applicable to welded wire fabric. The latter's resistance to slip is not a function of the contact area of the longitudinal wire in the concrete, on the contrary it is dependent on the anchoring ability of the transverse welded wires. The range of the sizes tested show that two welded transverse wires develop sufficient anchorage to develop the ultimate strength of the pulled-out wire.

INTRODUCTION

The tests were made under the sponsorship of the Wire Reinforcement Institute to determine the resistance to slipping of welded wire fabric when subjected to bending loads in concrete slabs or when pulled out of concrete blocks.

Pilot tests were first performed on two 4-in. thick slabs with a 60-in. span, followed by tests of two 3-in. thick slabs with the same span, all loaded for bending. SR-4 strain gages attached to the welded wire fabric were used to determine the stress distribution in the steel which indicated that slip was effectively prevented by the welded transverse wires.

Pull-out tests designed to measure the anchorage ability of the welded transverse wires were performed on 48 specimens consisting of various sizes of welded wire fabric embedded in blocks of concrete 3 in. thick, 15 in. wide and 14 or 20 in. long. Measured quantities were: pull-out load, slip at pull-out and free ends, and tension in the longitudinal wire between the two transverse wires. Samples of the fabric supplied for pull-out tests were tested for tensile strength and weld strength.

MATERIALS

Welded wire fabric

The steel was supplied by a member company of the Wire Reinforcement Institute and met the requirements of the Standard Specifications for Welded

*Presented at the ACI 48th Annual Convention, Cincinnati, Ohio, February 27, 1952. Title No. 48-45 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 8, Apr. 1952, *Proceedings* V. 48. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Aug. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Partner, Concrete Engineering Co., Tacoma, Wash.

Steel Wire Fabric for Concrete Reinforcement, ASTM designation A 185. The fabric used for slab bending tests was No. 2 wire, 0.2625-in. diameter, spaced 6 x 12 in. The average tensile strength of this wire was in excess of 90,000 psi. Fabric for the pull-out tests ranged in size from No. 00, 0.331-in. diameter, with a tensile strength of 84,000 psi to No. 6, 0.19-in. diameter, with a tensile strength of 95,000 psi. The specified minimum tensile strength for welded wire fabric is 70,000 psi, and the specified minimum yield strength is 56,000 psi.

Concrete

Concrete having a 5-in. slump was machine mixed and rodded when placed in the molds. The proportions were Type I portland cement—94 lb, Long Island sand, 0 to No. 4—240 lb, Long Island gravel, No. 4 to ¾ in.—280 lb, and mixing water—5.75 gal. The average 28-day compressive strength of the concrete was 3350 psi.

TEST SPECIMENS

Slab bending tests

Four slab specimens 34 in. wide and 66 in. in over-all length were cast (Fig. 1). The reinforcing mats were welded wire fabric with No. 2 wires

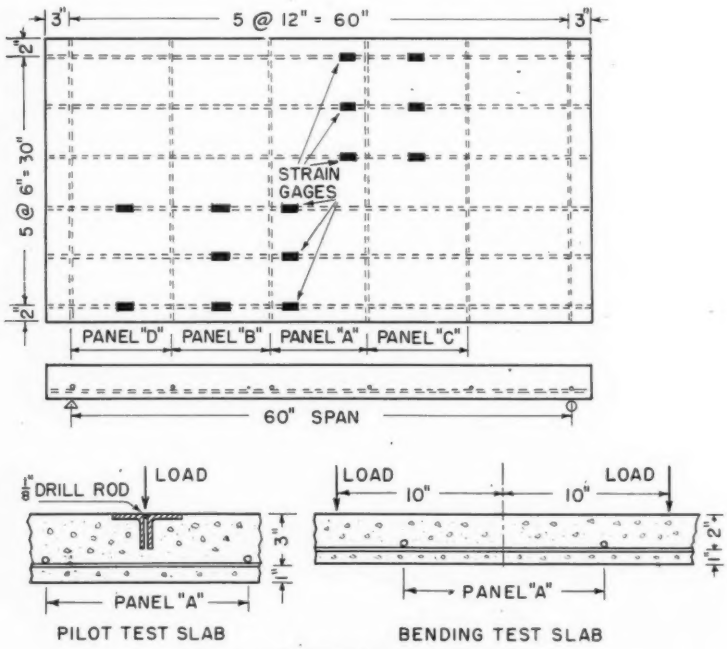

Fig. 1—Details of bending test slabs

spaced 6 x 12 in. The first pair of slabs were 4 in. thick, with the longitudinal wires placed at a depth of 3 in. One of these slabs (pilot test slab) was divided at the midspan section by a pair of 2 x 2-in. angles. Line contact between the angles was provided by placing a ⅛-in. diameter drill rod between the angles, in the plane of the top surface of the slab. This arrangement had three purposes; first, it provided a cleavage plane exactly through the midspan section of the slab; second, the tension in the longitudinal wires of panel A was readily determined from the simple leverage system of loading; and third, the system of loading closely simulated a pull-out test for the longitudinal wires. The other two slabs were cast with a thickness of 3 in., with the longitudinal wires placed at a depth of 2 in. Both slabs were loaded for bending, one with a concentrated load at the center, the other with loading applied equally at the third points, of the 60 in. span. The latter system of loading placed all the welded joints at the ends of the center panel, panel A, under the maximum bending stress.

SR-4 type A-7 strain gages were attached to the wires at the locations indicated in Fig. 1. The gages were Duco cemented to the wire as shown in Fig. 2, and held in place during drying by wrapping with several turns of fine thread around the gage. After making the soldered connection to the gage, the installation was covered with a paper sleeve and the space around the gage filled with Petrosene wax. Fig. 3 shows a welded fabric mat in the form with strain gages installed and ready to be embedded in concrete.

Pull-out tests

The 48 pull-out test specimens were cast in concrete blocks 3 in. thick and 15 in. wide and consisted of three specimens of each of 16 combinations of wire size and spacing. Nine combinations used a transverse wire spacing of 12 in., with a concrete specimen length of 20 in., and seven combinations having a transverse wire spacing of 6 in., and a concrete specimen length of 14 in. An SR-4 strain gage was applied to the middle longitudinal wire of each specimen to measure the amount of pull-out tension transmitted

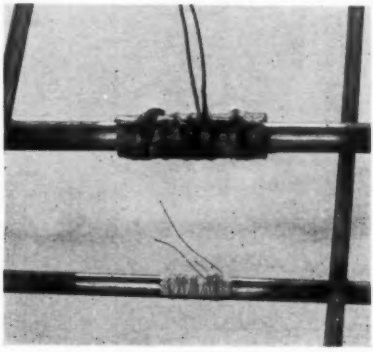

Fig. 2—Method of application of SR-4 gages to wire

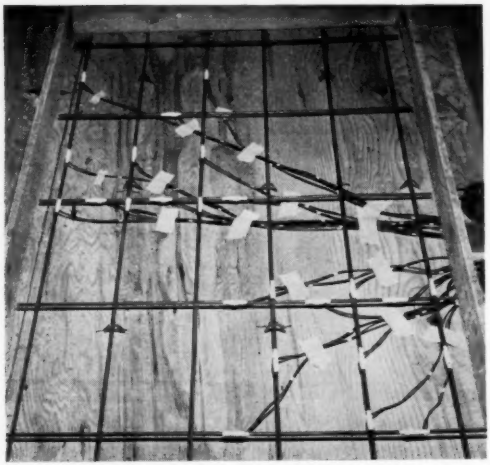

Fig. 3—Strain gage installation on welded wire fabric for bending test

beyond the first welded transverse wire (Fig. 4). The dimensions of the welded wire fabric in the pull-out tests are given in Table 1.

DISCUSSION AND ANALYSIS OF TESTS

Slab bending tests

Slab bending tests were made on the loading frame shown in Fig. 5. A 60-in. simple span was used for all four test slabs, with one end of the span supported on a round bar. The slabs were loaded by means of a hydraulic jack. To measure the jacking load, a bar with attached strain gages served

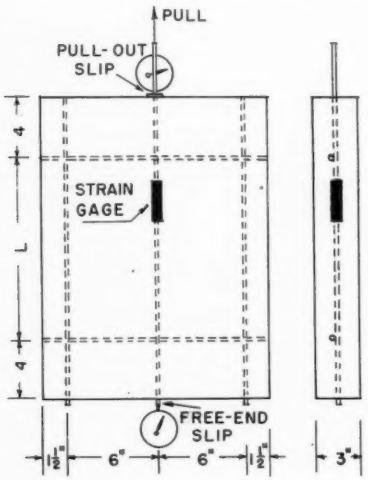

Fig. 4—Pull-out test specimen showing strain gage installation

TABLE 1—PHYSICAL PROPERTIES OF WELDED WIRE FABRIC, PULL-OUT TESTS

Series No.	Wire size		L, in.	Tensile strength, psi	Weld strength, psi	Weld strength / Tensile strength, percent
	Longitudinal	Transverse				
1	2	2	12	87,500	77,300	88
2	2	4	12	87,500	71,200	81
3	2	6	12	87,500	61,000	70
4	2	8	12	87,500	40,500	46
5	6	6	12	95,000	90,000	95
6	6	8	12	95,000	79,500	84
7	00	0	12	84,000	47,000	56
8	00	2	12	84,000	43,000	51
9	00	4	12	84,000	39,000	46
10	2	2	6	86,500	81,100	94
11	2	6	6	86,500	70,500	81
12	6	6	6	93,000	88,300	95
13	6	8	6	93,000	65,200	70
14	00	0	6	84,000	49,500	59
15	00	2	6	84,000	49,900	59
16	00	4	6	84,000	37,900	45

as a force indicator. The 4-in. slabs were loaded with a single concentrated force at the center of the slab. The first 3-in. slab was loaded at the center, and the second was loaded at the third points with equal concentrated loads. Strain reading were observed for each 500-lb increment of load. These first three slabs were loaded to destruction, fracturing the concrete and longitudinal wires at the middle of panel A, the midspan section. The strains indicated by gages in panel A exceeded the readings of panels B, C, and D by a ratio greater than the ratio of bending moment, suggesting that the welded intersections of the fabric prevented the longitudinal wires from slipping in the concrete. After loading to destruction, the concrete was removed and examination of the fabric showed all welds to be in perfect

Fig. 5—General arrangement for bending test

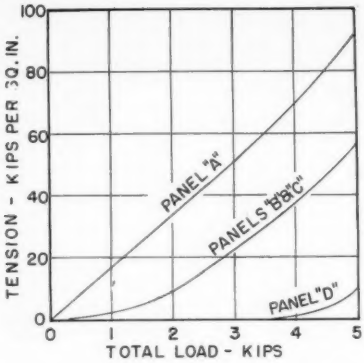

Fig. 6—Tension stress distribution on longitudinal wires, bending test

condition. The fourth slab was loaded at the third points so that all of panel A and parts of panels B and C were included in the zone of maximum bending stress. Thus the welded intersections of the two middle transverse wires with the six longitudinal wires were subjected to the maximum tension stress in the longitudinal wire. Two transverse cracks formed in the slab when the total applied load reached 1.5 kips. The cracks appeared at the boundaries of panel A close to the two middle transverse wires of the welded fabric. From the strain measurements plotted in Fig. 6, it can be seen that the 1.5-kip load produced an average tension amounting to 26,000 psi for the longitudinal wires of panel A, at the same time producing an average tension in panels B and C amounting to 5000 psi. The difference in tension, 21,000 psi, obviously was transferred to the anchorages provided by the transverse wires. As the load was increased, the tension stress difference between panel A and neighboring panels B and C continued to increase. At the 5-kip load, this difference amounted to 36,000 psi. At the same time, the tension difference between panel C and panel D was 46,000 psi. The slab fractured at a total applied load of 5.5 kips. All longitudinal wires failed in tension with typical cup-cone fractures. Although the fracture zone ran across the slab in the vicinity of the boundary between panels A and B, every weld in this highly stressed region remained in perfect condition. The fracture of this slab is shown in Fig. 7.

Fig. 7—Fractured section of bending test slab after loading to destruction. No welds failed in this test

Pull-out tests

The pull-out tests were performed using a pair of small hydraulic jacks coupled together with a short length of copper tubing to provide equal lifting force to each jack. A short beam attached to the heads of the two jacks contained the wire grip. The grip transmitted the load to the beam through a load cell utilizing electric strain gages. Dial indicators measured slip at both the pull-out and free ends. The test set up is shown in Fig. 8.

The slip resistance of welded wire fabric embedded in concrete is not proportional to the contact area between the steel and concrete. It is dependent on the ability of the welded transverse wire to provide anchorage. The concept of bond stress as determined by the investigations on plain and deformed bars would thus result in a fictitious bond-slip relationship of fantastic proportions. To illustrate this point, the curves of Fig. 9 were plotted. The values plotted were taken from measurements on pull-out test specimens of welded wire fabric embedded 14 in. in concrete. The curves show that for a slip of 0.005 in., bond stresses would range from 1000 psi to 2700 psi for smooth bars! The author denotes this concept for welded wire fabric as a psuedo bond stress.

A rational approach to the slip resistance of welded wire fabric in concrete requires a consideration of the efficiency of the welded transverse wires in anchoring the fabrics to the concrete. Thus the most important variables

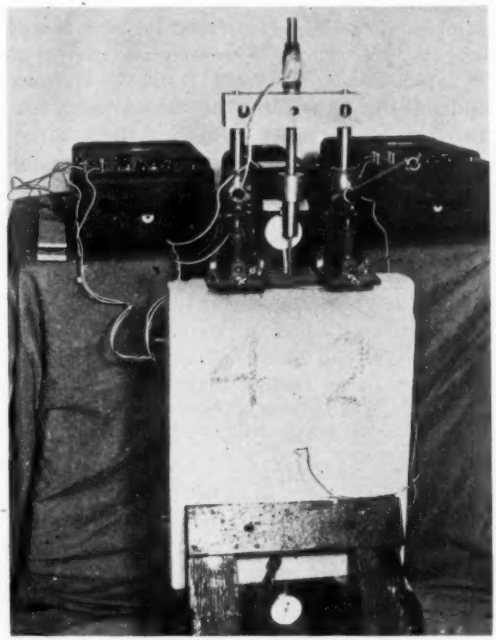

Fig. 8—General arrangement of pull-out test

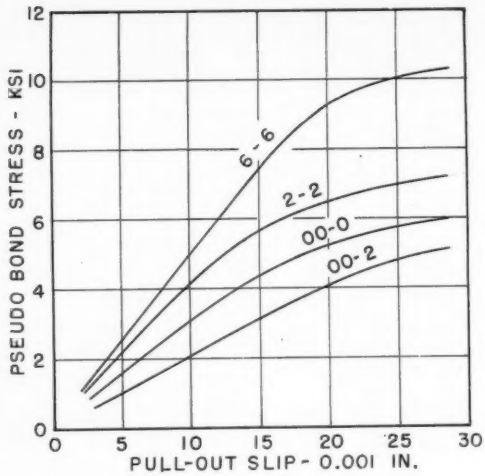

Fig. 9—Pseudo bond stress-slip curves for four sizes of welded wire fabric

are the size ratio between the transverse and longitudinal wire of the fabric and the quality of the weld connecting them. Approaching the problem from this point of view, it is necessary to determine the quality of the welds from samples of the fabric furnished for each test specimen. Thus the transverse wire of each sample is securely clamped in the jaw of a device called the "Carlton-Fitzpatrick Weld Tester." The force required to pull the longitudinal wire away from the welded transverse wire is measured, and compared with the tensile strength of the longitudinal wire recorded in Table 1. Note that the weld shear strengths for equal sizes of longitudinal and transverse wire ranged from 88 to 95 percent, and that for fabric having transverse wire six sizes smaller than the longitudinal wire, it was as low as 45 per cent of the tensile strength of the longitudinal wire.

In the pull-out tests, three measurements were recorded as a function of the pull-out load: (1) slip at the pull-out end, (2) tension in the longitudinal wire between the first and second transverse wire, and (3) slip at the free end. No free-end slip developed in specimens having transverse wires of equal size or two sizes smaller than the longitudinal wire. Only when the transverse wire was four sizes smaller than the longitudinal wire and the pull-out load approached the ultimate strength of the longitudinal wire did free-end slip start. When the difference was six sizes, the free-end slip began at an average pull-out level of 75,000 psi. Slip at the pull-out end was less dependent on the size ratio of the transverse and longitudinal wires up to pull-out tensions of 40,000 psi, however, at the higher load levels, the larger transverse wires offered greater resistance to pull-out slip. The effect of transverse wire spacing was relatively unimportant, the main difference being a slightly greater pull-out slip at higher stresses for the 12-in. spacing.

The most interesting measurements were the strain readings obtained from the gages attached to the embedded longitudinal wire between the first and second transverse wire. These measurements indicated that for fabric having transverse wires equal in size to the longitudinal wires or 2 sizes smaller, no tension was transmitted beyond the first welded transverse wire until the pull-out tension reached 40,000 psi. When the difference was four wire sizes, the first welded transverse wire prevented transmission of tension up to a pull-out tension level of 33,000 psi, and for a six wire-size difference, 18,000 psi. The slip and transmitted tension data are summarized by curves in Fig. 10 to 13. The tension transmitted beyond the first welded transverse wire is closely related to the wire size ratio. These data are best summarized by the curves in Fig. 14.

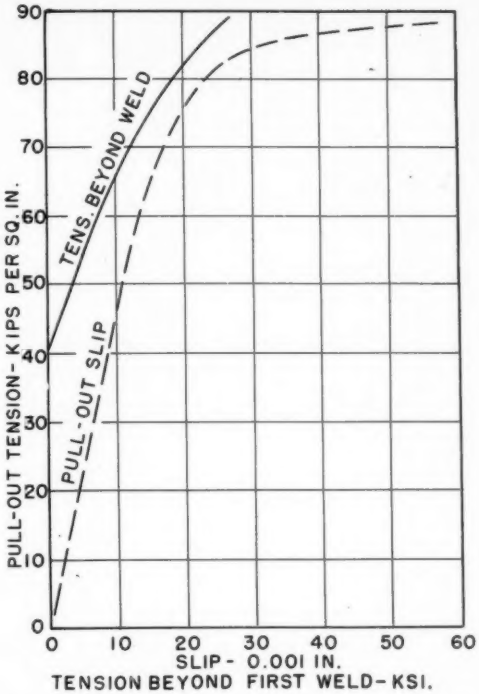

Fig. 10—Average slip and tension beyond first weld for fabric having transverse and longitudinal wires of equal size

These curves show that at the maximum design stress level, namely 30,000 psi, the first welded transverse wire offers 100 per cent anchorage for the longitudinal wire against slip up to four wire-size differences. For six wire-size size differences, the first welded transverse wire is capable of 82 per cent anchorage at the 30,000 psi stress level.

Of the 48 pull-out tests performed, all but two of the specimens failed in pure tension with well-known cup cone type of fracture (Fig. 15). The two weld shear failures occurred in the No. 00 wire with No. 4 transverse wire, and in both cases the pull-out load had virtually reached the ultimate tensile strength of the pulled-out wire before shearing the welds. The type of shear failure produced is shown in Fig. 16.

SUMMARY AND CONCLUSIONS

Welded wire fabric has slip-resisting properties proportional to the strength

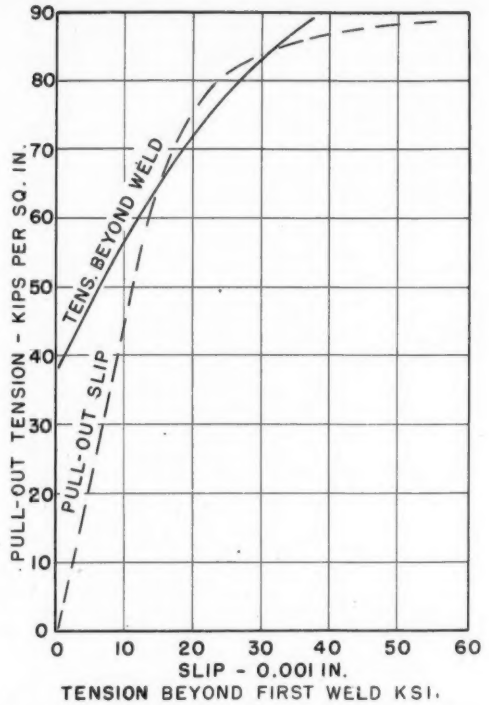

Fig. 11—Average slip and tension beyond first weld for fabric having transverse wires two sizes smaller than longitudinal

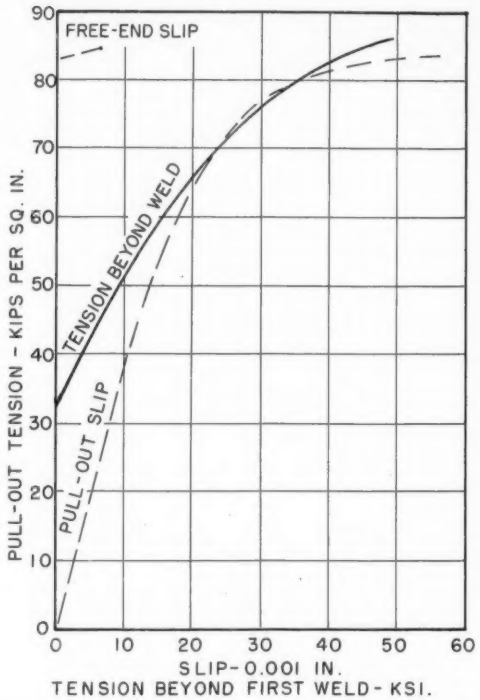

Fig. 12—Average slip and tension beyond first weld for fabric having transverse wires four sizes smaller than longitudinal

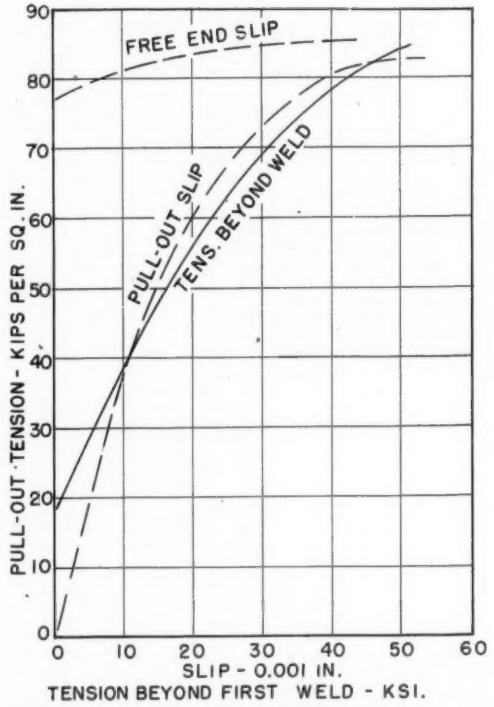

Fig. 13—Average slip and tension beyond first weld for fabric having transverse wires six sizes smaller than longitudinal

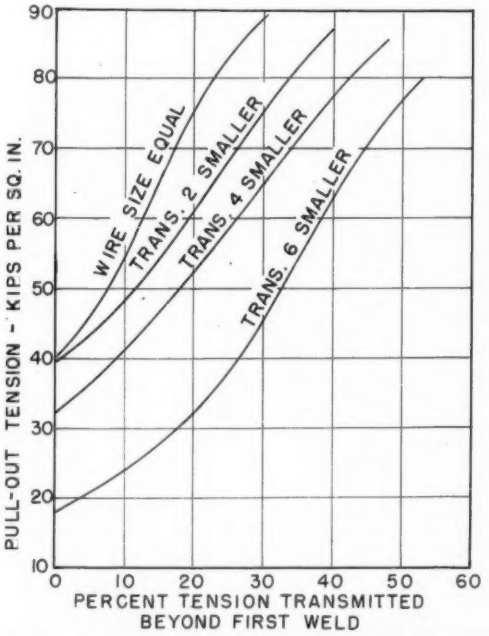

Fig. 14—Percentage of pull-out tension transmitted beyond first weld for four size combinations of wire

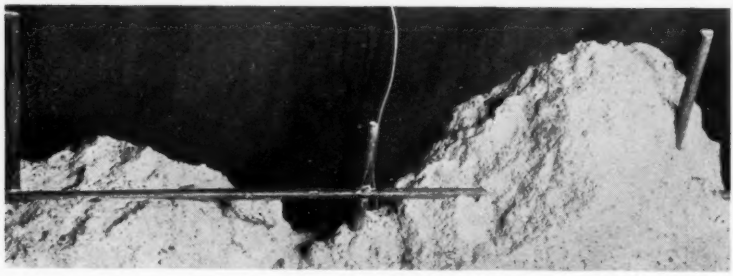

Fig. 15—Typical cup-cone tensile failure of longitudinal wire in pull-out tests

of the welded transverse wires. The term *bond* as applied to smooth or deformed steel bars does not define the resistance to slipping of welded wire fabric reinforcement; anchorage is a better term, because it defines the function of the transverse welded wires which are responsible for resisting slip.

Destructive load tests on concrete slabs reinforced with welded wire fabric demonstrated convincingly the mechanical anchorage qualities of the welded fabric. In every case, failures were pure tension, with all welds remaining intact.

In the pull-out tests, slip and transmission of tension beyond the first embedded transverse welds were proportional to the size decrement between transverse and longitudinal wires. For decrements up to four wire sizes, the first welded intersection provided sufficient anchorage to prevent any transmission of tension beyond the weld for pull-out loads up to 30,000 psi. Within the four wire-size decrement, two welded transverse wires proved capable of developing the tensile strength of the pulled-out longitudinal wire, and for the six wire-size decrement, seven out of the nine pull-out specimens developed the ultimate tensile strength of the pull-out wire, the two remaining specimens failed in shear when the pull-out load had virtually reached the tensile strength of the longitudinal wire.

From the results of the tests, the author concludes that no valid reason exists for requiring bond stress analysis of welded wire fabric reinforcement of the quality used in the tests. To guarantee proper anchorage, it therefore would be necessary to select the appropriate size ratio of transverse to longitudinal wire, and to specify minimum weld shear strengths.

Fig. 16—Weld shear and scraping of longitudinal wire, pull-out test

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Damage to civil engineer structures caused by the Imaichi earthquake (in Japanese)

SHUNZO OKAMOTO and KEIZABURO KUBO, *Transactions*, The Japan Society of Civil Engineers, No. 10, 1951, 1951, 14 pp.

A report on the investigation of the Imaichi earthquake which occurred Dec. 26, 1949. Deals mainly with damage to buildings, bridges and viaducts.

Proceedings of the First U. S. Conference on Prestressed Concrete

Prestressed Concrete Conference, Room 1-163, Massachusetts Institute of Technology, Cambridge 39, Mass. 256 pp. $1.62 in United States, $1.71 to foreign countries

Compilation of the 38 papers presented at the First U. S. Conference on Prestressed Concrete, August 14 to 16, 1951, of which ACI was a co-sponsor with other interested organizations. Papers deal with applications, materials, design and research pertaining to prestressed concrete.

Industrial furnaces V. 1

W. TRINKS, John Wiley & Sons, Inc., New York, 4th Edition, 526 pp. $10.00

Incorporates recent scientific facts about heat transfer as well as present day design and operating principles and techniques. Contains sections on capacity, economy, thermal efficiency, strength and durability, and movement of gases in furnaces. Tables have been brought up-to-date and many of the statements have been clarified. All calculations and tables have been changed from gross tons to net tons, in accordance with current usage.

Nine-story office building in London

Concrete and Constructional Engineering (London), V. 46, No. 12, Dec. 1951, pp. 359-363

Reviewed by GLENN MURPHY

A new office building in St. Giles' High Street, London, consists of a reinforced-concrete frame with brick walls. The columns and floor slabs were cast in place, while, in general, the beams and stair landings were precast. The building is 95 ft high and consists of four 48 ft 9 in. blocks separated by expansion joints. One roof at the first floor level involves two circular-arc reinforced concrete vaults each 36 ft wide; the remainder of the roofs are flat. Details of design and erection are given.

Prestressed concrete tanks at Hartlepool

Concrete and Constructional Engineering (London), V. 46, No. 12, Dec. 1951, pp. 365-370

Reviewed by GLENN MURPHY

Five tanks used in the extraction of magnesia from sea water have recently been completed at the Palliser Works of the British Periclase Co. Two of the tanks have a capacity of 2,000,000 gal. each and are said to be the first prestressed concrete tanks to be completed in Great Britain. The tanks are from 30 to 150 ft in diameter and from 36 to 46 ft deep. The wall thicknesses range from about 12 in. to 6 in. The Magnel-Blaton system of prestressing was used, and the prestress is such that the walls remain in compression for all depths of liquid. The prestressing procedure is described in detail.

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Concrete structures and corrosion

LESLIE H. GRIFFITHS, *Engineering* (London), Dec. 21, 1951, V. 172, No. 4482, pp. 775-777
Reviewed by ARON L. MIMSKY

General survey of the problems and methods of using protected reinforced concrete for acid tanks and similar purposes. Linings discussed include sheet rubber, brick, and synthetic resins; a portland cement-sand-natural rubber latex mixture applied as a smoothing and bonding base course is also described.

Paint manual

U. S. Bureau of Reclamation, First Edition, 1951. 204 pp. For sale for $1.25 by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C. and the Bureau of Reclamation, Denver Federal Center, Denver, Colo. Attention: 841

Included materials, techniques, and inspection of painting of all types of structural surface. Of particular interest is the brief discussion of paints for concrete walls and protective treatments for concrete floors. Dampproofing and waterproofing methods are also considered.

The continuous deep girder (Der statisch unbestimmt gela u. erte wandartige Trager)

H. BAY, *Beton-u Stahlbetonbau* (Berlin), V. 46, No. 3, Mar. 1951, pp. 60-64
Reviewed by RUDOLPH FISCHL

Deep girders with three or four supports are treated by the method of finite differences. Advantage is taken of the fact that the upper edge of a deep girder is practically unstressed. The results of photoelastic tests are shown to be in accordance with this theory. The author discusses further the practical design method, the reinforcement and different loading cases of this type of girder.

Construction of a dock in Cheshire

Concrete and Constructional Engineering, V. 46, No. 11, Nov. 1951, pp. 333-336
Reviewed by GLENN MURPHY

A 900-ft square dock is being built at Eastham, where the Manchester Ship Canal joins the River Mersey. The construction includes an approach lock 100 ft wide by 807 ft long and a leading wall 730 ft long. The concrete walls for the lock and dock are over 50 ft high. The principal feature of the article is the description of the systems of sliding forms and braces which were developed and patented by the contractors. The structures were cast in 4-ft lifts.

Joint, crack, and undersealing materials

Bibliography No. 12, Highway Research Board, Washington, D. C., Jan. 1952

The problem of preventing the infiltration of water, silt, sand, and other materials into joints and cracks in concrete pavements has concerned highway engineers ever since such pavements first came into existence. Bibliography 12, which consists of 122 selected annotated references dating from 1912 to 1951, was designed to readily acquaint engineers with the information available.

The law of cement hardening as a function of time (La loi du durcissement du ciment en fonction du temps)

J. M. JASPERS, *Revue des Materiaux de Construction* (Paris), No. 433, Oct. 1951, pp. 281-91
Reviewed by PHILLIP L. MELVILLE

Hardening of cement resulting from physicochemical changes, the strength R at a given age t is given by $R = K \log (1 + \log t) + K^1$ where K and K^1 are parameters. This was verified experimentally up to 90 days if R is known for at least 2 ages. Specifications were found to check the law except for German and ASTM (Types I and II) standards.

Complexometric analysis of iron, calcium and magnesium in cements and similar substances (Le dosage complexometrique du fer, du calcium et du magnesium dans les ciments et les substances analogues)

F.BECKER, *Revue des Materiaux de Construction* (Paris), No. 431-432, Aug.-Sept. 1951, pp. 248-53
Reviewed by PHILLIP L. MELVILLE

A chemical known as Complexion III is used for analysis of clinkers and cements. The product is the disodic ethylenediaminetetracetate which is derived from aminodiacetic acid. Detailed chemical information is given for quantitative analysis which can be done in about one hour. Fourteen references are addended.

Properties of plain and reinforced limerock concrete

S. L. BUGG and D. A. FIRMAGE, *Proceedings*, Highway Research Board, V. 30, pp. 217-231

Reports test results on approximately 1800 specimens of concrete in which limerock, a soft limestone common in Florida, was used as aggregate. Purpose of the tests was to determine the physical properties of limerock concrete in relation to conventional concrete.

Quantitative data are presented on bond, elasticity, thermal expansion, water requirements, compressive strength, and flexural strength. Also included are the results of a five-year performance survey made on an experimental limerock test road.

Floor elements in prestressed concrete (Elements de plancher en beton precontraint)

ROBERT BOURGES-MANOURY, *Communication* No. B 24, International Congress on Prestressed Concrete, Ghent, Belgium, Sept. 1951. 11 pp.

Deals with observations made in France on suspended floor elements and slabs of prestressed concrete which are produced industrially in several countries. After a description of the general characteristics of the two types of elements chiefly employed, the author discusses their practical use. Manufacturing methods are described and illustrated.

Heat treated concrete

P. CLARKSON, *Engineering* (London), V. 172, No. 4468, Sept. 14, 1951, pp. 345-346

Reviewed by ARON L. MIRSKY

Investigations into the effect of steam curing, both high- and atmospheric-pressure, on strength of concrete of various mixes.

Author concludes, among other things, that ordinary concrete may be successfully steam cured immediately after pouring, at an optimum pressure of 30 psi. The rate of initial temperature rise must be controlled to prevent cracking and obtain high strengths. However, steam curing reduces bond strength of reinforcing steel, necessitating mechanical bond.

Apparatus for measuring the water-cement ratio of concrete

Concrete and Constructional Engineering (London), V. 46, No. 12, Dec. 1951, p. 374

Reviewed by GLENN MURPHY

The device consists of a canister which is placed, open end down, in the freshly mixed concrete. A vibrator within the canister brings grout in contact with two electrodes, and the electrical resistance of the grout is measured. This value is compared with the value obtained from a batch having the same aggregate and the correct water-cement ratio. Differences of 0.05 in the ratio are discernible according to tests at the Building Research Station.

Lateral loads on high arches

JOHN GRAHAM WELSH, *Journal*, Institution of Civil Engineers (London), Dec. 1951, V. 36, No. 10, pp. 551-561

Reviewed by ARON L. MIRSKY

Calculation of transverse deflections and associated forces and moments in braced high open-spandrel arches. Effect of columns is taken into account.

Method is essentially that of Melan and Moorman; author sets up tabular scheme for numerical solution. An example is given.

This paper should be studied together with those by Michalos (ACI JOURNAL, Jan. 1951) and Baron and Michalos (ASCE *Proceedings*, Jan. 1951, Separate No. 51).

The factor of safety as applied to reinforced concrete design

PETER JOSEPH CARROLL, *Journal*, Institution of Civil Engineers (London), Nov. 1951, V. 36, No. 9, pp. 491-501

Reviewed by ARON L. MIRSKY

Author suggests reducing factor of safety for dead load from 2.2 to 1.5 while retaining the 2.2 factor for live load, in cases where dead load is fairly accurately known. He proposes accomplishing this by using 68 percent of the known dead load in calculations.

Suggestions are reminiscent of those of Boase (ACI JOURNAL, Feb. 1943, V. 39, pp. 277-296; *Engineering News-Record*, V. 128, No. 19, May 7, 1942, pp. 785-787) who suggested using 70 percent of the dead load.

Oceanographic instrument aids bridge engineers in pouring underwater concrete

WAYNE V. BURT and D. W. PRITCHARD, *Civil Engineering*, V. 21, No. 11, Nov. 1951, pp. 54-55

Reviewed by ROBERT E. WILDE

Brief discussion of surface and subsurface temperatures in both sea and fresh water bodies and their relation to placing underwater concrete. To determine subsurface water temperatures in Chesapeake Bay for underwater concrete placement in piers for the new Chesapeake Bay bridge, a bathythermograph proved invaluable for easy and accurate determinations. Surprisingly, even when surface temperature was slightly below 35 F (generally accepted safe minimum for placing concrete), that of subsurface waters, below about 20 ft, remained well above 35 F. Thus concrete could be placed continuously at subsurface depths.

Comprehensive bibliography of cement and concrete—1925–47

FLOYD O. SLATE, Engineering Experiment Station, Purdue University, Lafayette, Ind. 491 pp. $5.00

Over 40,000 references to much of the world's scientific literature on cement and concrete, covering the 23 years from 1925 to 1947. References are listed by subject and a complete author and chronological index is presented.

A partial list of the works consulted in the compilation includes *Chemical Abstracts, British Chemical Abstracts, Chemisches Zentralblatt, Engineering Index, Industrial Arts Index,* JOURNAL of the American Concrete Institute, *Proceedings* ASTM, *Transactions* ASCE, *Zement, Handbuch der Zementliteratur,* and *Das Schrifttum uber den Baustoff Beton, seine Herstellung und Verarbeitung.*

The deformation of concrete

E. FREYSSINET, *Magazine of Concrete Research* (London), No. 8, Dec. 1951, pp. 49-56

Attention is drawn to the lack of knowledge of the laws of deformation of concrete and examples are cited of slabs prestressed in two directions which have carried loads four or five times greater than those predicted by classical computation without cracks being formed.

To explain these and similar cases, the author develops his theory of the constitution of concrete; that is a complex "solid-liquid" to which the laws of thermodynamics may be applied. This enables general laws to be formulated which explain known experimental results and allow new ones to be predicted. In this way, shrinkage, creep, and—in general terms—elastic, plastic, instantaneous and time-dependent deformation of concrete are explained, as well as autogenous healing of cracked concrete. Deformation of concrete in shear is also discussed.

Designing for nuclear shielding

H. M. GLEN, *Civil Engineering,* V. 21, No. 11, Nov. 1951, pp. 51-53

Reviewed by ROBERT E. WILDE

Basic design data for construction of cells, barricades, and other forms of personnel shielding from radioisotopes. It is generally necessary only to take into account the gamma radiation. Solution of such problems can be divided into three steps, which are explained by the author.

Curves giving thickness of concrete and lead required to reduce intensity of gamma radiation by factor of 10 are presented. Three example problems for thickness determination are worked, using lead as the shielding material. The same procedure can be followed to solve design problems where concrete is used for shielding.

A contractor assays the specifications for Bull Shoals Dam

ROSS WHITE, *Civil Engineering,* V. 21, No. 11, Nov. 1951, pp. 32-33

Reviewed by ROBERT E. WILDE

A scrutiny of the actual cost of certain requirements of the plans and specifications for Bull Shoals Dam, and some suggested methods by which such costs can be reduced or eliminated on future projects. The author discusses costs and points out where savings could have been made in the project by revising such specification details as care and diversion of river, height differential limited to 20 ft, concrete lifts limited to 2½ ft, 5-day limit between successive lifts, refrigeration, and aggregates.

A major item mentioned was aggregates. Use of a distant quarry for coarse aggregates and manufactured sand added an estimated $4,250,000 to the cost as compared to use of natural sand and nearby quarry. The items discussed by the author add up to a possible saving of $7,200,000 for the Bull Shoals project.

Nomography and empirical equations

LEE H. JOHNSON, John Wiley & Sons, Inc., New York. 150 pp. $3.75

Considers the factors involved in the construction of nomographs, emphasizing a step-by-step progress from elementary to more complex factors. A method is introduced for analyzing an equation and its variables before plotting in nomographic form to determine the best arrangement of scales. Another innovation is a discussion of the effect of scale arrangement on the accuracy of a nomograph.

One chapter is devoted to the preparation of Z-charts for equations of three and four variables. Discussed also are parallel and perpendicular index lines, concurrent scales, recurrent variables and combined nomographs.

The second part of the book, empirical equations, takes up methods of curve fitting and curves of two, three and four constants.

Numerous detailed examples and problems for solution are provided throughout the book.

Concrete cracking. Part III—Flow through hydraulic structures

L. Boyd Mercer, *Commonwealth Engineer* (Melbourne), V. 39, No. 4, Nov. 1951, pp. 143-146

From a theoretical analysis of the flow of water through percolation paths in concrete structures, laboratory investigations, and field observations the author concludes that (1) routine permeability tests on laboratory specimens have little real value from the standpoint of permeability control of concrete water-retaining structures, (2) it is more important to design and test the concrete to reduce shrinkage than to produce a mix that will show a low coefficient of permeability in test specimens, (3) concrete used should be capable of being properly compacted with minimum segregation, (4) structural design must concentrate on minimizing internal and temperature stresses to avoid movement and cracking, and (5) practical aspects of the efficient treatment and grouting of construction and contraction joints are more important than theoretical considerations regarding permeability.

Design of two-way reinforced concrete slabs by the theory of plasticity (Berechnung Kreuzweise bewehrter Platten nach der Plastizitatstheorie)

H. Craemer, *Oesterreichische Bauzeitschrift* (Vienna), V. 6, No. 3, Mar. 1951, pp. 46-49

Reviewed by Rudolph Fischl

Under the assumption that the lines of rupture are straight, the moment along a line is constant and of maximum value, and the shear forces are zero. If the lines of rupture are known, the ultimate moment may be found easily by the principle of virtual work.

Two examples, solution of which by the theory of elasticity would be extremely laborous and involved, show the great advantage of this method. The first example gives the solution for a rectangular slab with a uniformly distributed load, two adjacent edges fixed and the others unsupported. The second example covers a triangular slab, fixed at the two short sides and unsupported at the hypotenuse, and loaded with a linear load along the hypotenuse.

Investigation of the suitability of Prepakt concrete for mass and reinforced concrete structures

Technical Memorandum No. 6-330, Waterways Experiment Station, Vicksburg, Miss., Oct. 1951, 172 pp. $1.00

Detailed description of an investigation designed to develop data on the suitability of Prepakt concrete for new construction work. A 72 cu yd block of low-portland cement content and a 15 cu yd reinforced concrete bridge pier of somewhat higher cement content were built. The study indicated that mass concrete having high strength and adequate durability can be made by the Prepakt method with a cement content of less than 2 bags per cu yd and that reinforced concrete of good quality can be made by this method with a cement factor of approximately 4 bags per cu yd. Sixty-two figures illustrate the progress of the tests and the appearance of cores drilled from the test structures. An appendix describes tests to determine suitable sand grading for Prepakt concrete.

Effect of mortar saturation in concrete freezing and thawing tests

Thomas M. Whiteside and Harold S. Sweet, *Proceedings*, Highway Research Board, V. 30, 1950, pp. 204-216

Based on laboratory freezing and thawing tests of 130 concrete beams from 31 batches of concrete, in which certain variables were controlled as closely as possible so that variations in durability could be related to variations in the water content and available air space in the mortar or paste component, authors conclude that when the paste-plus-air component has a degree of saturation of 0.91 or greater the concrete will deteriorate rapidly. All specimens in this range showed rapid deterioration.

All specimens with degree of saturation below 0.88 were highly durable although it is recognized that deterioration in this range may be caused by a variety of factors such as expansion of inferior aggregates, differential thermal expansion of concrete components, hydraulic pressure from resistance

to flow of water, nonuniform distribution of freezable water, or adverse chemical reactions.

Further conclusions are that investigations of factors such as different concrete materials or admixtures should be made with the degree of saturation about 0.80 so that results will reflect the influence of the variable being studied.

Why Type II cement?

F. H. Jackson, *Proceedings*, ASTM, V. 50, 1950, pp. 1210-1234

Comparative performance of eight typical Type I and five typical Type II cements in four of the projects of the Long-Time Study of Cement Performance in Concrete which have been exposed to severe natural weathering for periods up to 8 years is discussed. Condition of the structures in these four projects indicate that, up to the present, there is little choice between the two types as regards the influence of cement on durability. Greater differences were found between individual cements of both types than between the average values for each type. These facts indicate that, so far as the present indications of the Long-Time Study are concerned, Type II cement is not the answer to the problem of durability of concrete as affected by the cement. Concluding statement is that further research is necessary to determine to what extent variations in cement properties outside the range included in the present Long-Time Study program may affect durability.

Some experiments on the compaction of concrete by vibration

R. D. Davies, *Magazine of Concrete Research* (London), No. 8, Dec. 1951, pp. 71-78

Describes studies of cubes of very dry concrete of mix proportions suitable for small prestressed beams were compacted by vibration of various types, frequencies and amplitudes. Author concludes that (1) the effectiveness of vibration depends mainly on its acceleration, and not on the individual values of its frequency and amplitude; but very small amplitudes, about 0.002 in., are comparatively ineffective; (2) there is little to choose between vertical linear and horizontal circular vibration; vertical circular vibration is definitely better than either; and (3) the effectiveness of vibration increases with its acceleration, probably up to at least 20 g, but above about 12 g the increase is slow.

The total work done on a particle at a given depth was found to be related to the density at that depth. A connection was found between equivalent depth of compaction and the product of the amplitude of the beam and the number of vibrations per foot run. The depth of compaction increases with this factor but attains a limiting value, probably dependent on the concrete mix and the characteristics of the vibrator. It seems desirable, with vibrators designed to operate at constant acceleration, to operate them with a large amplitude and low frequency.

Effect of SO_3 on the alkali compounds of portland cement clinker

Terry F. Newkirk, *Journal of Research, NBS*, V. 5, No. 4, Nov. 1951, pp. 349-356

Reviewed by C. C. Fishburn

This investigation shows that the compound $Na_2O \cdot 8CaO \cdot 3Al_2O_3$, which is known to be one of the forms in which Na_2O may occur in portland cement clinker, is unstable at clinkering temperatures in the presence of SO_3. This compound reacts with SO_3 to form Na_2SO_4 and $3CaO \cdot Al_2O_3$. Any excess of Na_2O will remain as $Na_2O \cdot 8CaO \cdot 3Al_2O_3$, become a constituent of the clinker glass, or enter into solid solution with one of the other clinker components.

In clinker compositions containing both soda and potash together with SO_3, an alkali-sulfate phase is produced that contains both of the alkalies. For molecular ratios R_2O/SO_3 greater than unity, a preferential reaction of the SO_3 with K_2O is indicated. Empirical relationships between the alkalies and the SO_3 of the cement mixture indicate that the alkalies first combine with the SO_3 in the approximate molecular ratio $K_2O/Na_2O = 3$ to the extent that the composition limits of the mixture permit. The composition of the alkali-sulfate phase for mixtures of varying soda-potash contents with R_2O/SO_3 molecular ratios equal to 1, 2, and 3 have been determined. Equations are developed for calculating the approximate composition of the alkali sulfate and the potential phase composition of clinker-type mixtures having other R_2O/SO_3 ratios.

Why prestressed concrete?

LEO H. CORNING, *Civil Engineering*, V. 21, No. 10, Oct. 1951, pp. 41-45

Reviewed by ROBERT E. WILDE

Article based on paper presented by the author at the First U. S. Conference on Prestressed Concrete at Massachusetts Institute of Technology, August 1951.

Briefly reviews aspects of prestressed concrete in answering three basic questions: (1) Does prestressed concrete result in better structures? (2) Is it more economical? (3) Does it save critical materials in time of emergency or conserve natural resources over a long period? The author notes that complete answers to these questions will not be found except through further research and actual construction.

The author emphasizes the need for design specifications, even if only tentative, and predicts rapid progress in prestressing in this country if a design procedure and specification can be developed in the near future.

Unique laboratory houses human centrifuge

SARKIS M. BAGDOYAN, *Civil Engineering*, V. 21, No. 11, Nov. 1951, pp. 21-24

Reviewed by ROBERT E. WILDE

Unusual design and construction problems were successfully solved in the erection of the Aviation Medical Acceleration Laboratory, U. S. Naval Air Development Center, Johnsville, Pa. The primary purpose of the centrifuge apparatus in the building is to study the effects of high accelerations on humans and to test equipment for high-speed aircraft and rocket projectiles. The equipment will develop a force of 40 *G* in 6.85 sec.; the centrifuge mechanism weighs 84,000 lb.

Since the action of the centrifuge device is rotational, it was decided to construct a circular building of reinforced concrete, in addition to a concrete foundation for the 180-ton centrifuge motor.

Conservative design of the motor foundation was necessitated by tremendous dynamic forces developed by the centrifuge. It was necessary that the concrete remain a solid mass without any cracking caused by great differences in temperature between interior and exterior during hydration of cement; the area of steel was 0.1 percent of the area of concrete. A base ring transfers the 2000 ft-kip torque from the motor into the foundation by shear keys, rather than directly by bolts.

All areas in the building containing delicate instruments are shielded with copper from conductive and radiated electrical interference. Typical protection is that in the centrifuge chamber where the walls are covered with sheet copper; the floor is similarly shielded except that there is a 3-in. wearing coat of concrete on top of the copper.

Large reservoir designed for mining subsidence

Concrete and Constructional Engineering (London), V. 46, No. 12, Dec. 1951, pp. 353-358

Reviewed by GLENN MURPHY

A feature of the 4,000,000-gal. reservoir being constructed for the Corporation of Nottingham is that it is designed to resist failure from anticipated settlement as coal is removed from below the structure. Vertical settlements of 3 ft and a mean gradient of differential settlement of 1 in 180 are expected. The reservoir, which is 230 x 220 ft and 16 ft deep, is divided into 25 independent structural units each 46 x 44 ft. The units are connected only at floor level, where hinges are provided to permit relative rotation. The walls of each unit are independent and separated from the walls of the adjacent units by an initial gap of 4 in. Internal inspection chambers are provided at each wall joint.

The structure rests on a 4-in. plain concrete slab which in turn rests on a 10-in. bed of clay puddle. The latter is to reduce the stresses due to sagging, hogging, and horizontal movement of the ground. The floor consists of three thin slabs, so designed to increase flexibility and to reduce cracking.

Earthfill dams (Les barrages en terre)

C. MALLET and J. PACQUANT, Eyrolles, Paris, 1951, 340 pp., 2500 fr.

Reviewed by G. L. BLANCHARD

This appears to be, to the reviewer's knowledge, the first book published in French language entirely devoted to design and construction of earth fill dams. It must be pointed out that the authors have been guided and inspired by the works of American engineers and the studies of prominent United States specialists of soil mechanics on this particular type of hydraulic structure. Professor Terzaghi of Harvard Uni-

versity was kind enough to write an introduction to the book and has also made "in fine" some very pertinent notations.

The first chapter deals with the practical considerations leading to the choice of that type of dam, and the general requirements as regards the foundation subsoil, together with the incidental difficulties to be overcome. The second chapter stresses the necessity of providing adequate spillway structures designed to bypass the mass of the dam under the most unfavorable flood conditions. Equations and graphical methods are evolved, corresponding to various characteristics of the contemplated structures.

The third chapter and the fourth deal with infiltrations and risk of seepage and leakage. Formulas and methods of adequate design are examined and justified through original mathematical analysis of the physical behavior of the materials present.

The extensive fifth chapter is devoted to soil mechanics considerations as regards the stability of earth slopes constituting the mass of the dam. The equations evolved lead to sound methods of design which will help eliminating risky and obsolete empirical considerations. The concluding chapter six deals with the selection of adequate constitutive materials and the description of practical construction methods.

It may be worth noting that this book is the work of two members of the staff of scientists, technicians and engineers which are struggling incessantly and successfully in French North Africa to improve the living standards of rapidly increasing populations through engineering achievements, despite all difficulties arising from natural and local conditions.

Applying basic engineering principles to concrete unit manufacturing

M. W. Loving, *Rock Products*, V. 55, No. 1, Jan. 1952, pp. 232-236 & 254

Reviewed by Donald M. Agrimson

Describes how recent developments in the use of semi-dry mixes, prestressing and curing contribute to improved concrete units and structures; what has been said and done in the past has not been overlooked. Credit is given to men like Arthur N. Talbot, W. K. Hatt, F. E. Turneaure, M. O. Withey, Duff A. Abrams, and Harrison F. Gonnerman, for their basic research in and development of reinforced concrete in this country.

The reason why semi-dry concrete mixtures have not been used extensively in this country is that equipment used to mix and place concrete is designed for the wet or plastic mixes. Semi-dry concretes must be mixed in a paddle-type concrete mixer. Mixing must be two stage—first, approximately a two-minute dry-mixing period for the cement, sand, and coarse aggregate; second, after adding the water the concrete should be mixed at least two minutes.

The Rocla roller-suspension method is now being used in this country to produce water pressure pipe of reinforced prestressed as well as reinforced concrete of conventional design. One concern processed concrete on this type of machine with compressive strengths ranging from 8200 to 11,300 psi from a mix having three sacks of cement per 9.789 cu ft of concrete.

The Freyssinet process for producing prestressed pressure pipe is a complicated and expensive method; but concrete of extraordinary strength and density can be produced by the use of it. Freyssinet conducted a test on a pipe, made under his process (shell thickness 1.97 in., 31.5-in. diameter, and 19.68 ft long) in which it was subjected to an internal hydrostatic pressure of 1000 psi without leakage.

Swedish methods and equipment designed to manufacture reinforced concrete piles are being used and widely accepted in this country. The Swedish external vibrators used by one concern were seen to consolidate, in eight minutes, the semi-dry concrete in a master mold forming 14 x 14-in. x 10½-ft piles. The same cross-sectional dimension pile in 40-ft lengths was processed in less than an hour, using 10 external vibrators; this time covered the complete cycle of placing the concrete, vibrating, and removing the master mold.

After observing concrete prestressing in this country and reading information from Sweden revealing the extensive and successful use in that country of prestressed structural units of all kinds in more than 500 industrial buildings, the author is firmly convinced that an extensive use of prestressing should be made in this country to alleviate the shortage of structural steel.